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Structural Assessment of Bridges with Deck Hinges

AM-STR-06021

June 2014

AM Asset Management & Maintenance

Standards

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Document Attributes

TII Publication Title	*Structural Assessment of Bridges with Deck Hinges*
TII Publication Number	*AM-STR-06021*

Activity	*Asset Management & Maintenance (AM)*
Stream	*Structures (STR)*
Document Number	*06021*

Document Set	*Standards*
Publication Date	*June 2014*
Historical Reference	*NRA BA 93*

NRA DMRB and MCDRW References

For all documents that existed within the NRA DMRB or the NRA MCDRW prior to the launch of TII Publications, the NRA document reference used previously is listed above under 'historical reference'. The TII Publication Number also shown above now supersedes this historical reference. All historical references within this document are deemed to be replaced by the TII Publication Number. For the equivalent TII Publication Number for all other historical references contained within this document, please refer to the TII Publications website.

An tÚdarás um Bóithre Náisiúnta
National Roads Authority

Volume 3 Section 1
Part 3
NRA BA 93/14

Structural Assessment of Bridges with Deck Hinges

June 2014

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Summary:

This Advice Note provides guidance on the structural assessment of deck hinges and presents a strategy for the management of National Roads Authority bridges with deck hinges.

Published by National Roads Authority, Dublin 2014

VOLUME 3 ROAD STRUCTURES: INSPECTION AND MAINTENANCE

SECTION 1 INSPECTION

PART 3

BA 93/14

STRUCTURAL ASSESSMENT OF BRIDGES WITH DECK HINGES

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1 INTRODUCTION

General

1.1 This Advice Note is concerned with guidance on the structural assessment of deck hinges. The management strategy for the National Roads Authority is included at Annex F.

a) The scope of this document includes all types of bridges with hinge deck details as illustrated in Annex B and described in 1.2 and 1.3.

b) Information has been included on the use of non-destructive testing (NDT) techniques (see Annex F1.15 to F1.19).

c) Guidelines on the structural assessment of hinges, based on experimental results from the Highways Agency in the UK, are provided (see Annex F1.10 to F1.14 and main text in Advice Note).

e) Remedial options are identified (see Annex F2).

f) Timescales have been outlined (see Annex F1.2 and F2).

1.2 A typical hinge detail (similar to that shown in Annex B) consists of a narrowing of the concrete section in flat slab or beam and slab decks to form a throat, through which steel reinforcement passes between the cantilever and suspended span. There are known variations to this arrangement, with differing construction sequences, details of reinforcement, types of reinforcement, skews and edge details, particularly where service bays are included. It is important for assessment to establish the reinforcement configuration and type, and if any reinforcement is misplaced or damaged.

1.3 Structures with a different detail known as a Wichert truss are also to be included within the scope of the strategy. These structures were used in areas subject to significant differential settlement such as mining subsidence.

1.4 In Ireland the NRA Eirspan Bridge Management System database has been developed to allow input and management of data associated with the hinge deck strategy.

2 ASSESSMENT METHODOLOGY

Background

2.1 Hinge joints were introduced into bridge decks as a means of simplifying the design and standardising details on bridges having a range of span and functional requirements. They were also used to mitigate the effects of movements due to temperature, creep, shrinkage and differential settlement. The disadvantages of hinge joints are that they are not easily accessible for inspection or maintenance due to their form, and being mostly located over or under live traffic lanes. They are vulnerable to deterioration in the event of bridge deck waterproofing failure, where chlorides seeping through the joint can cause reinforcement corrosion. This reinforcement is crucial to the integrity of the joint, and the loss of reinforcement section, or associated concrete spalling can induce higher stresses, leading to eventual failure by yielding. It has been noted during inspections carried out on behalf of the Highways Agency in the UK, that the majority of hinges have cracks running through the full depth of the throat, indicating that the hinge has been, or is subject to tension and may not be working as originally intended. This complicates any structural assessment of the hinge, and also brings into question the fatigue endurance of the reinforcement.

Structural Strength

2.2 This Advice Note presents guidelines for the assessment of deck hinges. Assessment is carried out to check the structural adequacy of the hinge and its ability to carry the specified traffic loading. It should be carried out in two parts:

1. structural analysis: to determine the range of load effects on the hinge joint;

2. analysis of hinge: to calculate the capacity of the hinge in its current condition.

2.3 Structural analysis is performed assuming that the hinges behave as pinned supports. Analysis should consider the normal local load effects specified by NRA BD 21 such as accidental wheel loads, thermal, and differential settlement, and the presence of particular design features such as cantilever pipe bays, skew and dog-leg joints. Skews greater than 10° should be taken into account in the structural analysis. Limited testing in the UK has indicated that skew does not affect the strength of a hinge and the methodology presented in Chapter 3 can be used.

2.4 One of the objectives of the assessment is to identify a deterioration trigger point to feed into a monitoring and inspection regime to assist in determining when interim measures are required. To facilitate this, a sensitivity analysis should be carried out to determine the influence of variations in the condition of the structure. Defects can be categorised under loss of throat reinforcement cross-section, reinforcement yielding, misplaced reinforcement, concrete debonding, and loss of link reinforcement. A range of severity of each defect (and any other factors) should be considered, and the position of the structure within this range determined. For the sake of consistency of reporting, sensitivity should be expressed in terms of ‘usage factor’, defined as the ratio of the ultimate load effect to the assessed hinge capacity. Technical Acceptance procedures in accordance with NRA BD 02 will apply to this assessment work.

Fatigue

2.5 A fatigue assessment should be carried out. Fatigue failures occur under the frequent application of quite small stress ranges due, typically, to the passage of individual vehicles. Fatigue damage accumulates under repeated loading and the rate of damage accumulation is very sensitive to the stress range experienced at the fatigue site as the load passes.

2.6 In order to perform a fatigue check on the bars at the hinge, it is necessary to evaluate the stresses generated at the fatigue site. A simplified methodology for doing this is presented in Chapter 3.

Preliminary Assessment

2.7 The assessment methodology outlined in Chapter 3 entails a degree of complexity that may not be required in many cases. It is recommended that preliminary analysis using simplifying assumptions be carried out as this may be sufficient to demonstrate adequate strength. As a first approximation, the shear capacity of the hinge can be taken as the vertical component of the yield strength of the tension and compression hinge bars. A check should be carried out to ensure that there is sufficient shear steel to carry the force in the compression hinge bars. This will yield a conservative estimate of the shear capacity of the hinge, provided due allowance is given to the condition of the reinforcement in and around the hinge (see Clauses 3.9 – 3.11).

Detailed Assessment

2.8 If the adequacy of the joint cannot be demonstrated using the simplified assumptions of 2.7, then the full procedure outlined in Chapter 3 should be carried out. An example calculation is presented in Annex D.

2.9 Alternative methods can be used provided it is demonstrated that the hinge behaviour is adequately modelled. This shall be agreed with the NRA in the Technical Acceptance procedures in accordance with NRA BD 02.

3 STRENGTH ASSESSMENT

Strength Mechanism

3.1 Shear force is carried through the hinge by the following force components whose effects are additive (see Figure 3.1):

1. The tension in the hinge bars (*T*): these bars contribute a vertical force equal to the resolved vertical component of their yield strength.

2. The compression in the hinge bars (*C*): These bars contribute a vertical force equal to the resolved vertical component of their yield strength. This component acts on the deck shear truss as a vertical load through the intersection of the centrelines of the diagonal bars and the top or bottom reinforcement of the deck. Because it is usual in hinge reinforcement systems for the tension and compression bars to match each other in number, size and angle, and because it is assumed that compression yield equals tension yield, the horizontal force component of the compression hinge bars matches that of the tension hinge bars. Together they generate no net longitudinal force through the hinge.

3. Diagonal concrete compression: This component only acts if there is some horizontal compression force at ULS (*H*) acting across the hinge. At ULS there can be an internal force reacting against the horizontal restraint provided by the dowel bars acting at yield. The horizontal force may also have an external source, such as the horizontal reaction generated in a deck due to inclined columns or mass abutments. This situation is discussed below. It is likely that this component of the strength of the hinge is limited by the strength of the link and C bar system in the deck on both sides of the hinge.

3.2 Dowel action may also contribute to the strength of the hinge but will only be fully mobilised after the diagonal concrete compression strut has failed. Because of the complexities involved, it is recommended that dowel action be ignored.

Solution Procedure

3.3 Iteration is needed to find the strength available in the system. A straight, diagonal compression strut is considered to act through the hinge, at an angle θ to the horizontal (see Figure 3.1). The width of the strut, w, is governed by the geometry of the hinge and the angle θ. The maximum force in the strut (V_C) is limited by the available horizontal restraint H, and the compressive strength of the concrete. Thus the vertical component of the strut force is the lesser of $H \tan\theta$ and $w\, b\, \sigma_C \sin\theta$, where b is the width of the cross section and σ_C is the compressive strength of the concrete in the hinge. The strength σ_C is taken as 0.67 times the cube strength. The vertical force is assumed to act through the point where the diagonal force meets the mid-depth of the horizontal top or bottom reinforcement in the deck. The strut force is also limited by the available vertical steel which should be sufficient to carry the combined force in the strut and compression hinge bars.

3.4 The solution can be found using the following procedure. Consider a vertical dividing plane through the deck some distance from the hinge and find the resultant vertical force due to all the effective links and C-bars between the plane and the hinge, acting at yield. Note that links are only considered effective if they enclose the tension and compression hinge bars and they extend for at least half the depth of the concrete section. Subtract from this resultant the vertical force needed to resist the compression hinge bars. The line of action of this resultant is calculated: this defines the angle θ of the strut force. The position of the dividing plane is adjusted until there are just sufficient links to carry the diagonal compressions from the concrete and the compression hinge bars. The contribution from the concrete diagonal compression is V_C for the associated value of θ. The total shear capacity is V_C plus the vertical component of the tension and compression hinge bars.

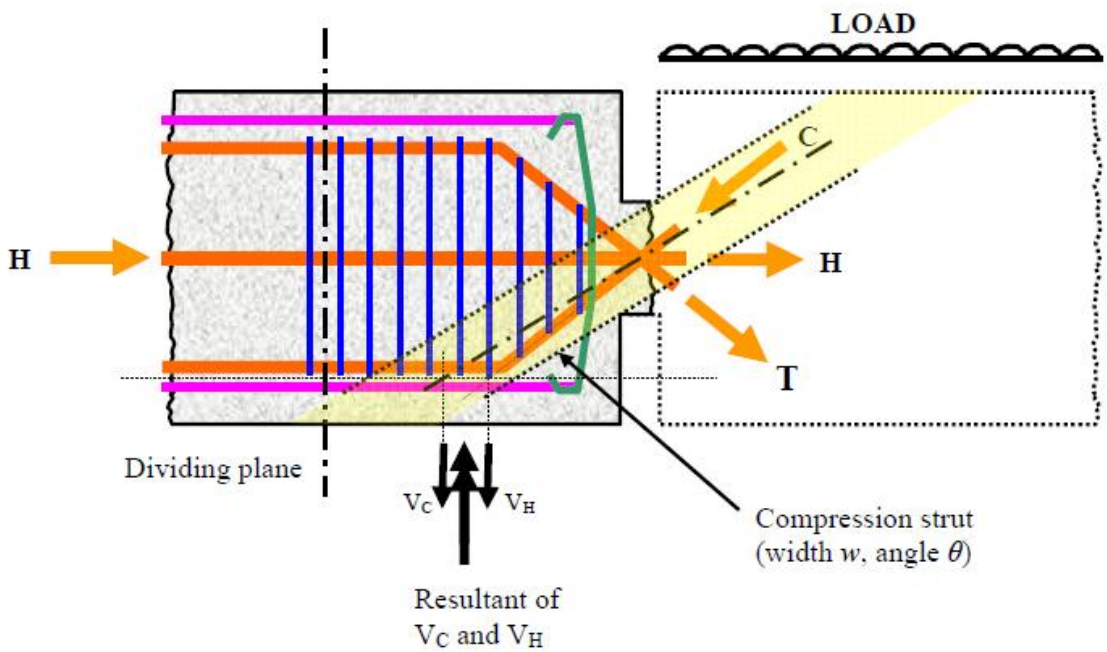


Figure 3.1 Forces Acting in and Around the Hinge.

External Horizontal Restraint

3.5 External forces which induce compression across the hinge can increase the available horizontal restraint. Judgement is needed to assess what external force can be included in the value of H used in the assessment of V_C. It should be a force that can be relied upon to be present at ULS. It is not possible to provide advice on the value of the force that might be present. Each case should be examined individually. Where doubts exist, beneficial forces should be ignored as a conservative measure.

3.6 There may also be external influences which reduce the value of H, such as the tension force across the hinge due to the sum of a number of bearing friction forces or thermal shrinkage. In this case it is likely that, because the diagonal compression strut is pushing outwards, the friction forces would be reversed, or at least neutralised, at ULS. Where the magnitude of the tension force can be estimated, this should be subtracted from the internal restraint provided by the dowel bars. Where significant tension forces are suspected, due for example to the presence of a full depth crack through the throat, a conservative approach is to ignore the contribution of the concrete compression strut.

Allowance for Faulty Construction

3.7 The magnitude of the diagonal concrete compression component is sensitive to the accurate positioning of the formers used to make the hinge. If there is specific data from inspections on the geometry of the hinge this can be used directly in the calculations. If working from drawings all the dimensions of the hinge should be modified by 10% so as to minimise the strut width *w*.

3.8 The hinge bars may be misplaced within the hinge. Test data shows that this does not have a great effect on capacity and does not need to be considered in assessment.

Allowance for Deterioration

3.9 If inspection indicates that the reinforcement may be corroded, allowance should be made in the assessment as for other reinforced concrete elements. The analysis allows different steel section losses due to corrosion to be applied to the tension hinge bars, the compression hinge bars, the dowel bars and the vertical steel reinforcement.

3.10 If the concrete in the hinge is damaged in places the value of σ_c may be reduced pro rata with the proportion of the hinge concrete that cannot be relied upon. If the throat concrete is seriously damaged, it may be necessary to omit the compression strut component completely.

3.11 Testing has shown that local debonding of the reinforcement in the hinge does not affect the capacity.

Fatigue Assessment

3.12 Fatigue can develop in the tension hinge bars at cracks in the hinge concrete. The fluctuating stresses in the bars have two main components:

- component due to fluctuating shear across the hinge. This can be evaluated by simple statics, taking account of any longitudinal restraint;
- component due to hinge rotation.

3.13 Under shear load the concrete in the hinge cracks. The characteristic pattern is that cracks develop from the corners of each hinge former at a right angle to the adjacent tension hinge bars. As in all cracks in reinforced concrete, the bar debonds for a short length each side of the crack. If the hinge is overloaded in shear the debonded length increases. When a rotation is applied to the hinge the cracks open and close slightly. The stress change in the bar at the hinge depends on the debonded length: the longer the debonded length, the smaller the resulting stress change in the bar.

3.14 Under long-term cyclic loading, as the crack system around the hinge develops, the debonded length increases with deterioration of the bond and/or the application of excessive loads to the bridge. When assessing an existing bridge it is not possible to know the stress history of the hinge bars with any degree of accuracy. The following procedure is based, conservatively, on stresses in the bars in a young, newly cracked structure. Some of the parameters depend on the geometry of the hinge details, but they are relatively insensitive to differences in the geometry. Because the prediction process is inexact, universal coefficients are used which are based on the geometry of the typical hinges studied.

3.15 The fatigue assessment presented below is based on a finite element analysis and takes account of the stresses induced in the hinge bars due to shear and rotation following the formation of shear cracks. There is some concern that particular hinge details and/or loading history may result in a crack pattern, such as vertical cracking in the hinge throat, which differs from that assumed and which may lead to high local stresses in the hinge reinforcement.

3.16 A test programme is being carried out in the UK which will examine these local stresses and, in doing so, further consider misplacement of the diagonal bars and provide an additional overview of global hinge behaviour. Associated fatigue testing is also being carried out of reinforcement bars in bending to radii smaller than those covered by current codes of practice.

3.17 Pending the results of this test programme, it is recommended that the method presented here be used with caution and the assessment conclusion be considered as provisional only.

Procedure for Assessing Fatigue Failure

3.18 The scissor bars are assessed for fatigue in the same way as any other reinforcement bar subject to significant cyclic loading. For the fatigue analysis the local stress in the scissor bar is taken as:

$$\sigma_{SB} = 0.30 \times \sigma_S + K \times a \times h/D$$

where:

σ_S is the axial stress in a tension scissor bar assuming all the shear force across the hinge is carried by the tension scissor bars;

K is a parameter taken to be 0.30 MPa;

α is the difference between the rotations of the deck on the two sides of the hinge;

D is the nominal diameter of the scissor bars;

h is the depth of the hinge, measured as the clear depth between hinge formers.

Note: σ_{SB} includes stresses due to bending of the bar as well as stresses due axial force in the bar.

3.19 From the analysis of the bridge structure, an influence line for σ_{SB} along the lines of the loading lanes can be generated. These influence lines combine the effects of shear and rotation and can be used to assess directly the fatigue life of the bar.

Allowances for Faulty Construction of the Hinge

3.20 The fatigue assessment is insensitive to faulty construction of the hinge.

3.21 If inspection indicates that the reinforcement may be corroded, allowance should be made in the assessment as for other reinforced concrete elements. Because it is fatigue which is being checked this should include a stress concentration factor as well as allowing for loss of section. Note that the loss of section allowance only applies to the shear component. The stress due rotation is a compatibility stress and is not affected by loss of section.

3.22 If the concrete in the hinge is damaged then more of the shear will be carried by the tension scissor bars and, at the same time, the stresses due to rotation will be reduced. As an additional check the fatigue in the tension scissor bar should be analysed assuming the following expression for σ_{SB}:

$$\sigma_{SB} = 0.55 \times \sigma_S + 0.20 \times K \times a \times h/D$$

3.23 If corrosion is present, allowance should be made for reduced reinforcement area. Debonding of the reinforcement reduces the stress concentration in the reinforcement and can be ignored.

4 REFERENCES

4.1 National Roads Authority Design Manual for Roads and Bridges (NRA DMRB)

NRA BD 02 Technical Acceptance of Structures on Motorways and Other National Roads

NRA BD 21 The Assessment of Road Bridges and Structures

NRA BD 27 The Protection and Repair of Concrete Road Structures

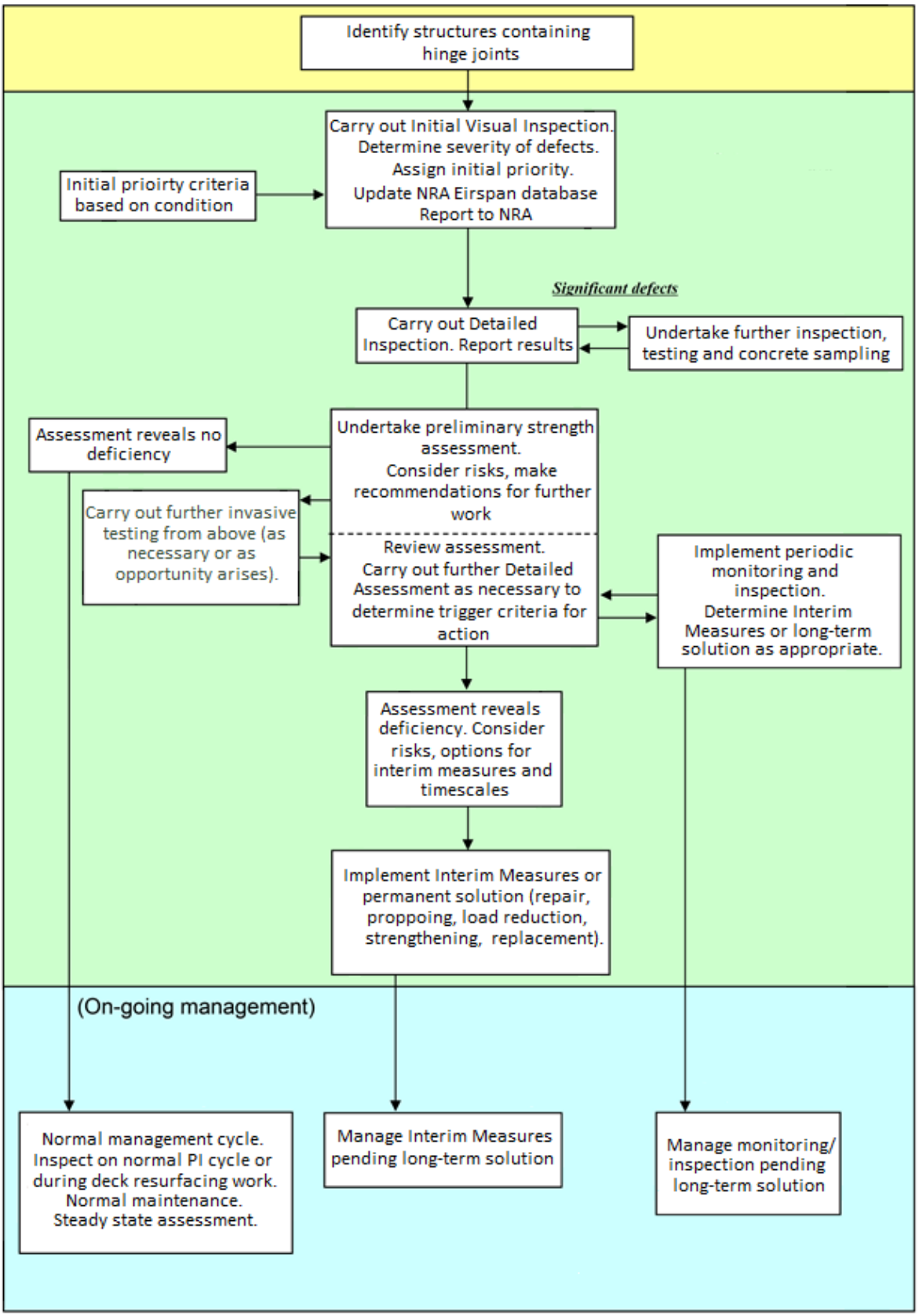
5 ENQUIRIES

5.1 All technical enquiries or comments on this document or any of the documents listed as forming part of the NRA DMRB should be sent by e-mail to infoDMRB@nra.ie, addressed to the following:

"Head of Network Management, Engineering Standards & Research
National Roads Authority
St Martin's House
Waterloo Road
Dublin 4"

……
Pat Maher
Head of Network Management,
Engineering Standards & Research

ANNEX A MANAGEMENT STRATEGY FLOWCHART



ANNEX B TYPICAL HINGE DETAILS

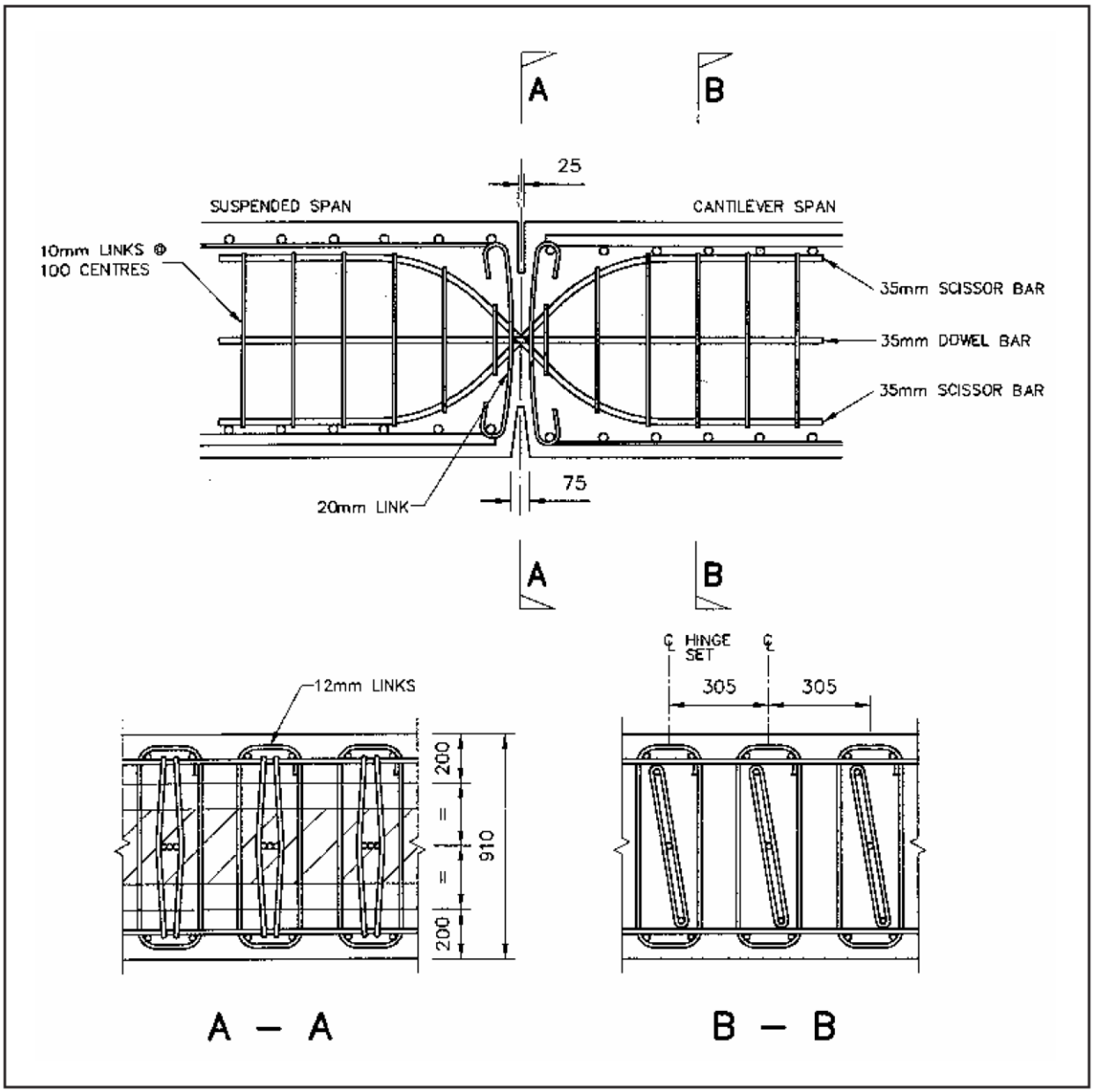


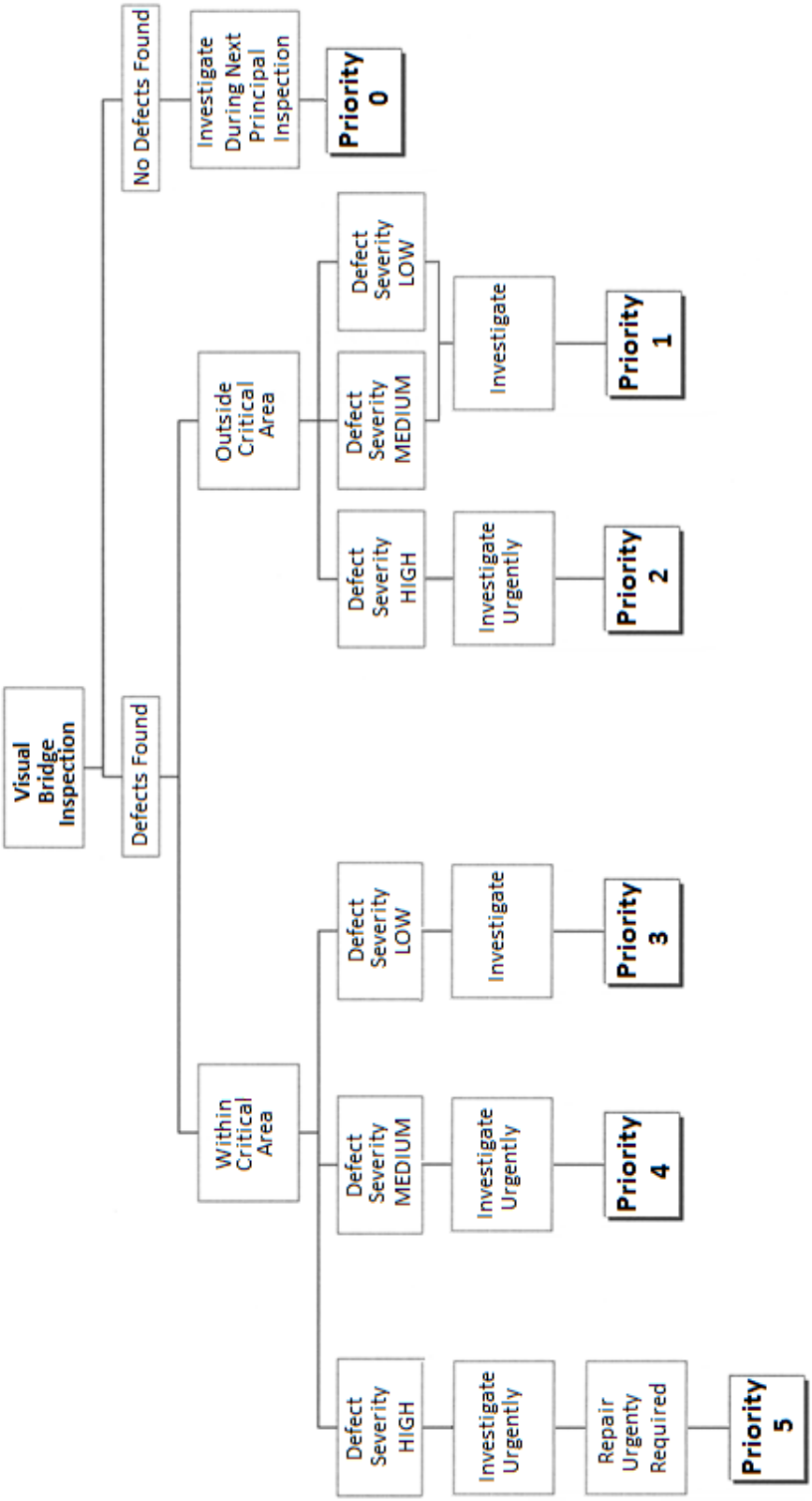
Figure B.1 Typical Hinge Details

ANNEX C: INITIAL PRIORITISATION

C.1 The initial prioritisation presented in the flowchart below is based on the external condition of the hinge. This provides a useful way of allocating resources early in the management programme, which can be modified as further information is obtained. Other factors that need to be considered are:

- other aspects of the inspection/assessment of the structure;
- long-term management strategy for the bridge;
- consideration of current or future road improvement schemes.

PRIORITY SCORING FLOWCHART



ANNEX D EXAMPLE CALCULATION

D.1 To illustrate the application of the procedure for determining the shear strength of a deck hinge, the example shown in Annex B, Figure B.1, is used. The hinge reinforcement consists of high tensile bars, diameter 35mm and yield strength 450N/mm^2. The concrete strength is taken as 31N/mm^2. All partial factors are taken as 1.0. The throat is 305mm deep and 25mm wide. Vertical reinforcement consists of two 20mm C-bars at 50mm from the centre of the hinge, 10mm links at 100mm centres around the tension and compression hinge bars (starting at 75mm from the hinge centreline), and 12mm links at 20mm centres around the top and bottom reinforcement (starting at 75mm from the hinge centreline), for each set of hinge bars. The concrete section considered in the analysis is 915mm deep x 915mm wide, and contains three sets of hinge bars and associated vertical reinforcement.

Steel details	**Strength per leg (kN)**	**Strength (kN)**	**Acting at (mm)**
3×2 No 20mm C-bars	78.5	471.2	50.0
4×3×2 No 10mm links	19.6	471.2	425.0
3×3×2 No 12mm links	28.3	508.9	275.0
Total	**-**	**1,451.4**	**250.6**

Table D.1

D.2 Assessment consists of choosing a vertical plane at some distance from the hinge and determining the vertical force resultant due to all the vertical steel (links and C-bars) between this plane and the hinge acting at yield. For example, if the plane is taken at 600mm from the centre of the hinge, then the following steel contributes:

Note that the first two 10mm links are ignored as they do not extend more than half the depth of the concrete section.

D.3 This gives the total capacity of the vertical steel (S_c) and its line of action measured vertically from the centre of the hinge. The vertical component of the force in the three compression hinge bars (V_h) is 784.8kN. The centre of the hinge bars is located at 70+12+32/2 = 98mm from the face of the beam, assuming that the cover to the 12mm links is 70mm. Thus the compression force is acting at (915/2 - 8) tan45° = 359.5mm from the centre of the hinge.

D.4 As the combined compression force in the concrete strut and the compression hinge bars should be carried by the vertical steel, the vertical component of the strut force (V_c) can be determined by subtracting the vertical component of the compression hinge bar force from the vertical steel capacity.

Steel details	Strength per bar (kN)	Vertical component (kN)	Acting at (mm)
Vertical steel	-	1,451.4	250.6
Compression hinge bars (3x2 No 32mm)	370.0	784.8	359.5
Resultant	-	666.6	122.5

D.5 The maximum strut force that the vertical steel can carry (in addition to the force in the compression hinge bars) is 666.6kN acting at 122.5mm. The angle θ that this strut makes with a horizontal plane is $\tan^{-1}(361.5/123.2) = 71.2°$. The width of the compression strut, determined from this angle and the geometry of the hinge throat, is given by $(305\text{-}25 \tan \theta) \cos\theta = 74.7$mm.

D.6 The force in the strut (C) is calculated from $666.6/\sin\theta$ to be 704.3kN. However, this assumes that its magnitude is limited by the available vertical reinforcement. However, it may also be limited by the available horizontal restraint. In this case, horizontal restraint is provided only by the three 32mm horizontal dowel bars passing through the throat (it is assumed that there is no external source) which is calculated as 1,109.9kN. The horizontal component of the strut force is $953.7\cos\theta = 227.2\text{kN} < 1{,}109.9\text{kN}$: thus the horizontal restraint is not limiting.

D.7 A further check is carried out to ensure that the compression strength of the concrete is not exceeded. For this plane, the maximum possible strut force is $915\times74.7\times0.67\times31 = 1{,}419.9\text{kN} > 953.7\text{kN}$: thus the concrete compression strength is not limiting. This indicates that the strut force of 704.3kN is correct, and the shear capacity of the hinge given by the vertical component of the concrete strut + tension and compression hinge bars = 704.3 + 784.8 + 84.8 = 2,236.2kN.

D.8 This analysis is then repeated for different plane locations until the optimum location is determined which yields the maximum shear capacity. This is most conveniently done with a spreadsheet. For each assumed plane, the shear capacity of the hinge is determined from the lower value of:

- the available vertical steel capacity plus the vertical component of the tension hinge bars;
- the vertical component of the strut (limited by either the available horizontal restraint or the compressive strength of the concrete) plus the vertical component of the compression hinge bars plus the vertical component of the tension hinge bars.

The actual capacity is then determined from the plane which maximises the shear capacity.

D.9 The summary calculations shown in Annex D.2 indicate that at a position of 800mm (or 850mm) from the hinge (where the C-bars, 36 legs of the 10mm links and 24 of the 12mm links are effective), the shear capacity is 2,641.5kN, with the limiting factor being the vertical capacity of the contributing vertical steel. However, the limit imposed by the horizontal restraint is not too different 2,734.9kN. In this case, therefore, it would be important to ensure that the condition of the links and dowel bars is adequate.

D.10 For comparison, the shear capacity provided solely by the vertical component of the tension and compression hinge bars is 1,569.6kN. Thus the detailed analysis increases the assessed capacity by 68%.

D.11 The effects of deterioration in the hinge can be taken into account by modifying the section properties and geometry appropriately. For example, a 5% loss of steel section in the hinge reinforcement due to chloride induced corrosion would reduce the shear capacity of the hinge to 2,596.2N. Note that different corrosion losses can be applied to the hinge bars, dowels and links.

Position of plane		400	500	600	700	750	800	850	900	1,000
Vertical strength of links:										
C-bars	No. of participating legs	6	6	6	6	6	6	6	6	6
	Strength	471.2	471.2	471.2	471.2	471.2	471.2	471.2	471.2	471.2
	Acting at	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0	50.0
Links1	No. of participating legs	12	18	24	30	30	36	36	42	48
	Strength	235.6	353.4	471.2	589.0	589.0	706.9	706.9	824.7	942.5
	Acting at	325.0	375.0	425.0	475.0	475.0	525.0	525.0	575.0	625.0
Links2	No. of participating legs	12	18	18	24	24	24	24	30	30
	Strength	339.3	508.9	508.9	678.6	678.6	678.6	678.6	848.2	848.2
	Acting at	175.0	275.0	275.0	375.0	375.0	375.0	375.0	475.0	475.0
TOTAL	Sc =	1,046.2	1,333.6	1,451.4	1,738.9	1,738.9	1,856.7	1,856.7	2,144.1	2,261.9
Acting at		152.5	222.0	250.6	320.8	320.8	349.6	349.6	420.1	449.0
Vertical force in comp bars	Vh = C cos 45	784.8	784.8	784.8	784.8	784.8	784.8	784.8	784.8	784.8
Acting at		359.5	359.5	359.5	359.5	359.5	359.5	359.5	359.5	359.5
Vertical compnent of strut	Vc = Sc - Vh	261.4	548.8	666.6	954.1	954.1	1,071.9	1,071.9	1,359.3	1,477.2
(must be positive, otherwise compression bars cannot be carried)										
Acting at		0	397.1646	122.5	289.0	289.0	342.4	342.4	455.0	496.5
Angle	θ	90.0	42.2	71.2	51.2	51.2	46.4	46.4	38.3	35.9
Width of strut (take as zero if negative)		0	209.346	74.7	171.6	171.6	192.2	192.2	223.8	232.4
Force in strut	C (limited by vertical steel)	261.4	817.8	704.3	1,224.1	1,224.1	1,480.2	1,480.2	2,192.7	2,518.7
Horizontal component of struct force		0.0	606.3	227.2	766.9	766.9	1,020.9	1,020.9	1,720.5	2,040.0
Available horizontal restraint		1,109.9	1,109.9	1,109.9	1,109.9	1,109.9	1,109.9	1,109.9	1,109.9	1,109.9
Vertical struct force limited by hoz restraint		1.812E+19	1,004.6	3,257.0	1,380.8	1,380.8	1,165.3	1,165.3	876.9	803.6
Max possible strut force	C (limited by concrete strength)	0.0	3,958.7	1,412.8	3,244.9	3,244.9	3,635.3	3,635.3	4,232.4	4,394.2
Vertical strut force limited by comp strength		0.0	2,656.6	1,337.3	2,529.1	2,529.1	2,632.5	2,632.5	2,623.8	2,577.1
Shear capacity governed by:	Horizontal restraint	1.812E+19	2,574.191	4,826.6	2,950.4	2,950.4	2,734.9	2,734.9	**2,446.5**	**2,373.2**
(strut+tension+ compression bars)	Concrete strength	**1,569.6**	4,226.2	2,906.9	4,098.7	4,098.7	4,202.0	4,202.0	4,193.4	4,146.7
	Links	1,830.9	2,118.4	2,236.2	2,523.7	2,523.7	**2,641.5**	**2,641.5**	2,928.9	3,046.7
	Capacity	1,569.6	2,118.4	2,236.2	2,523.7	2,523.7	**2,641.5**	**2,641.5**	2,446.5	2,373.2

Shear capacity	**2,641.5 kN**

Table D.2 Results of Assessment

ANNEX E RISK ASSESSMENT

E.1 Introduction

E1.1 In order to develop a strategy for the maintenance and repair of bridges containing concrete hinges, a methodology is required to rationally assess the comparative risks that may arise from the deterioration process. The procedure presented here was developed from that developed for half-joints, as described in the Highways Agency Interim Advice Note IAN 53/04.

E1.2 All costs quoted in this annex are based on the Highways Agency standard BA 93/09 and are given in GBP.

E1.3 Although there is no single set methodology for qualitative risk assessment, the practice is well established in a number of industries. Qualitative risk assessment is being used increasingly by managers of infrastructure assets and some published guidelines are available. The guidance within CIRIA Report SP125 'Control of risk: A guide to the systematic management of risk from construction' has generally been adopted in this methodology. It should be noted that there are no right or wrong answers in qualitative assessment, only relative opinion. The principal value of qualitative risk assessment is not necessarily in the final ranking outcome but in the process of risk identification. It is a formalised process enabling work to be reported objectively and open to scrutiny.

E1.4 The definition of risk is widely accepted as being the product of the probability or likelihood of an event occurring and the consequences arising from the event:

Risk = Likelihood of occurrence × Consequence

E1.5 A simple numerical scale can be used for the likelihood and consequence. It is important to stress that the indicator may have no numerical significance, other than to show qualitatively that one asset is likely to require more management effort than another.

E.2 Outline Methodology

E2.1 A number of factors have been identified which may increase or decrease the likelihood of a bridge with deck hinges becoming substandard, as follows:

P1: Configuration and Access;
P2: Assessed Capacity;
P3: Current Condition;
P4: Rate of Deterioration;
P5: Future Loading.

E2.2 It is important to establish a numerical scale that may be used objectively for each of these five factors. The scale adopted for the likelihood is based on CIRIA SP125 five point scale:

Very Low	1
Low	3
Medium	5
High	7
Very High	9

E2.3 Not all factors should be given equal weighting and therefore a significance factor should be applied to further enhance the assessment. A distorted numerical scale has been adopted to take account of the potential difference between very high and very low significance as follows:

Very Low	0.5
Low	1
Medium	2
High	4
Very High	8

E2.4 The significance factors are used to weight the relative likelihood factors.

E2.5 The consequences arising from a bridge collapse due to the failure of a deck hinge, in terms of potential loss of life and/or confidence in this form of bridge construction, would be so great as to totally dominate any qualitative risk assessment. The safety of the road user is paramount and it is a primary objective that all bridges with deck hinges be managed so that safety is assured. Therefore, consequences are considered solely in terms of the financial costs of investigation, assessment, repair and traffic delay costs.

E2.6 To enable the future management effort to be identified and readily grouped, a continuous numerical scale of 1 to 9 has been established for the cost consequence. Unlike the likelihood of failure, the indicator for consequence has a meaningful relationship to actual cost.

Consequence Factor		**Cost**
Very Low	1	£25,000
	2	£50,000
Low	3	£100,000
	4	£200,000
Medium	5	£400,000
	6	£800,000
High	7	£1,600,000
	8	£3,200,000
Very High	9	£6,400,000

E2.7 A distorted scale of costs has been adopted with each increase in consequence of 1 unit representing a doubling of cost. The consequence factor may be determined directly from the cost by the equation:

$$(\log_n(Cost/£25{,}000/\log_n 2)) + 1$$

E2.8 Alternatively, the cost may be determined from the consequence factor by the equation:

$$£25{,}000 \text{ x } 2^{(Consequence\ factor\ -1)}$$

E2.9 For example, a cost of £235,700 would have a consequence factor of:

$$(\log_n(£235{,}700/£25{,}000/0.301)) + 1 = 4.2$$

E2.10 Values up to £25,000 will have a consequence score of less than 1.

E.3 Likelihood of Occurrence

E3.1 The qualitative assessment of the likelihood of the hinges becoming substandard is determined by considering the five factors P1 to P5.

P1: Joint Configuration and Access

E3.2 The different generic arrangements of deck hinges identified during the initial data collection process are illustrated in Annex B. Ease of access for inspection is influenced by the joint arrangement.

E3.3 All deck hinges are difficult to assess: for this reason a mean value of 5 is assumed, modified as follows:

If access from above is not from live carriageway	-2
If access from below is from bearing shelf	-3
If access from below is over live motorway carriageway:	+2
If access from below is over minor road:	+0
If access from below is over river:	-2
No of hinges:	
Difficult access to more than one joint	+2
Difficult access to one joint	+1
Moderate	0

P2: Assessed Capacity

E3.4 The capacity of the joint can be determined according to the methodology presented in the Advice Note. A comparison between this assessed capacity and the capacity required to carry the assessment load indicates the degree of risk of failure of the hinge. This is most conveniently carried out by determining the usage factor, defined as the ratio of the ultimate load effect to the assessed hinge capacity. Where the usage factor is found to be greater than 1.0, it is likely that loading restrictions will be in place. Nevertheless, this increases the likelihood that the hinge will fail in the future.

E3.5 A median value of 5 is initially assigned to P2. The probability of failure is increased by 4 units for structures with usage factor greater than 2.0. The adjustments to be applied for assessed capacity are as follows:

Usage factor > 2.0	+4
Usage factor > 1	+2
Usage factor not known	0
Usage factor = 1	-2
Usage factor < 1	-4

E3.6 Where comparisons are borderline, i.e. the usage factor is just less than or just greater than 1.0, the accuracy of the assessment should be considered. Assessments that have incorporated assumptions of a dubious nature (in terms of condition of the hinge for example) should be considered less accurate and reliable and P2 should be varied accordingly.

P3: Current Condition

E3.7 Information on current condition should be based on the latest inspection report (or special inspection report carried out as part of this strategy) and where possible in relation to a Stage II assessment condition factor. For deck hinges in a fair condition, a median value of 5 is assumed with the following adjustment made for good and poor condition:

Poor	+2
Fair	0
Good	-2

E3.8 If particular concerns or defects have been identified which may affect the performance of the joints a further +2 adjustment may be warranted. Such defects might include spalling of concrete around hinge, cracking through the hinge, evidence of leakage, sag or deformation of the deck. If repairs have been undertaken a negative adjustment may be appropriate to reflect the long-term improvement in condition. If repairs are only cosmetic then no adjustment is warranted.

Specific defects	+2
Cosmetic or no repairs	0
Structural repair	-2

P4: Rate of Deterioration

E3.9 A median value of 5 is assigned, modified as appropriate. Direct measurements of concrete properties such as concrete permeability, chloride contamination, cover, etc, are not currently widely available for the majority of deck hinges. However, there are other indicators which can give an insight as to whether the likely rate of deterioration will be greater or lesser than the average.

E3.10 The type and condition of the road surfacing and seal in the hinge will influence how much salt is likely to penetrate through the deck. Due to poor maintenance in the past, a median value for P4 of 5 is assumed. A deck hinge in poor condition is likely to result in cracking at the throat and chloride contamination of the hinge reinforcement. Depending on road joint condition the following adjustments are appropriate:

Poor	+1
Fair	0
Good	-1

E3.11 The level of salt use on a route is an important consideration as this is a major contributor for the deterioration of reinforced concrete structures. The following adjustments are adopted depending on the salt usage:

High	+1
Medium	0
Low	-1

P5: Future Loading

E3.12 Increased usage and congestion on a route will increase the probability of a deck hinge becoming substandard and so increase the rate of deterioration of road joints. Consideration should be given to current and future road widening schemes. Routes which are likely to experience unchanged and average traffic growth are assigned a median value of 5. Urban and strategic routes are likely to see greater increases in future loading and traffic volume than rural routes. Access roads are less likely to see any increase in loading. As a guide the following factors are appropriate: however, local knowledge should prevail.

Motorway	+2
Dual A P Trunk Road	+1
Single Carriageway Trunk Road	0
Lane/Local Road	-2
Access Road/Footway	-4

E.4 Significance Factors

E4.1 Not all contributing factors should be given equal weighting. The significance factor applies a weighting to the likelihood. For the risk assessment of deck hinges, the relative significance given to each factor can be taken as follows:

P1: Configuration and Access	2
P2: Current Capacity	4
P3: Current Condition	4
P4: Rate of Deterioration	2
P5: Future Loading	1

E4.2 Management effort will be greatest for those bridges deemed to be imminently substandard. The current capacity (P2) and current condition (P3) of a joint will be the primary factors affecting whether or not a deck hinge is likely to be substandard at the present time and are given a 'high' significance score of 4. For those bridges deemed to be of adequate capacity but actively deteriorating, management effort will be required to prevent further deterioration but this may be spread over a number of years. Factors (P1) joint configuration and access, and (P4) rate of deterioration, are factors which generally indicate the potential for a deck hinge to become substandard in the future and are assigned a significance factor of 2. Future loading (P5) is considered to be of low significance as future increases in loading can be planned for well in advance of any potential problems arising and is assigned a significance score of 1.

E.5 Cost Consequence

F5.1 The overall costs of repair comprise the design costs, the actual costs of undertaking repairs and the cost to the road user in terms of traffic delays. Traffic delay costs are often many times greater than the actual cost of repair and should be taken into consideration when considering the impact of a structure becoming substandard. For structures with a calculated likelihood factor of 6 or greater, the Agent is required to estimate the costs of undertaking repairs to the hinges. If the Agent has a clear understanding of the remedial measures to be adopted and a detailed estimate of repair is available (inclusive of user delay costs) these costs should be reported.

E.6 Repair Costs

E6.1 The repair techniques suitable for deck hinges are limited and cost estimates may assume the implementation of a particular repair technique to assess the relative consequences of a structure becoming substandard. It is important to note that this is a comparative exercise using limited data. The total works costs include an allowance for access and traffic management costs. For bridges crossing a river or other watercourse access for repair by scaffolding off the deck may be assumed. For bridges over roads, access may be assumed to be via scaffolding from the road below.

E6.2 Generally traffic management will be required for repair from both above and below the deck. The nature of repairs is such that two running lanes are likely to be closed with contraflow running. The length of traffic management for contraflow may be assumed to be 5km for motorways and National roads, to accommodate cross-over points at an assumed distance of 3km. For single carriageway roads, traffic signalling with shuttle flow may be assumed. The time to undertake repairs is likely to be split say 75% from below deck and 25% from above deck and this would be reflected in the relative access and traffic delay costs incurred from above and below deck working.

E6.3 For underbridges over rail, access may be assumed to be by scaffold access tower and additional rail protection staff will also be required. Gaining access to a railway requires careful planning and liaison with the rail authorities to obtain track possessions. This will limit the time available to undertake repairs and every opportunity should be made to limit the works duration undertaken from below deck. In this case the time to undertake repairs is more likely to be split 25% from below deck and 75% from above deck.

E6.4 For specialist repair techniques the ratio of design and contract preparation costs to works costs will be relatively high and may be assumed to be as high as 50% of the contract value for each bridge (which includes traffic management and access costs).

E.7 Traffic Delay Costs

E7.1 Traffic user delay costs can be calculated using the computer program QUeues And Delays at ROadworks (QUADRO). Tables contained in the Highways Agency Trunk Road Maintenance Manual (TRMM) – Volume 1 have been derived from QUADRO to estimate traffic delay costs for different scenarios of traffic management restriction. These tables have been used as the basis for deriving the traffic delay costs per day.

E7.2 The traffic delay costs are related to the type of road, the degree of the restriction, the daily traffic flow, the percentage of Heavy Goods Vehicles (HGV) and the physical length of the works on site.

E7.3 The duration of the works above and below deck needs to be considered to obtain the total traffic delay costs. To evaluate traffic delay costs it is generally necessary to obtain the following information:

- road classification;
- the likely lane restriction;
- annual average daily traffic (AADT) flows;
- percentage of HGVs using the structure;
- alternative routes for diversion if appropriate;
- whether or not works are undertaken off-peak.

E7.4 The type of repair or investigation will dictate the nature of the lane restrictions for each road classification.

E7.5 In the absence of more local knowledge Table 4 presents typical traffic delay costs per 8 hour working day for repair. The percentage of HGVs which use the road influence the traffic user delay costs. Motorways are assumed to have 30% HGVs, National roads 20% HGVs and single carriageway roads are assumed to have 10% HGVs. Motorway slip roads are classified as wide single carriageways: where two motorways join, the percentage HGVs is more likely to be 30%. However, the difference in the costs for 10% and 30% HGVs is minor and the assumption of 10% for all situations is considered acceptable for the level of accuracy required.

E7.6 For minor roads with a width between 5.5m and 7.3m and two marked lanes the maximum traffic flow is assumed to be 5,000 AADT. For access roads the costs are assumed to be half those given for single carriage ways.

E7.7 For repairs, two running lanes are assumed to be closed with contraflow running. The length of traffic management for contraflow is assumed to be 5km. For single carriageway roads, traffic signalling with shuttle flow is assumed. The traffic management proposed is such that traffic is unlikely to divert onto alternative roads and therefore, no additional factors have been applied to the Highways Agency TRMM tables.

E7.8 Off-peak or night working is considered practical for most short duration repair work. The traffic delay costs presented in the Table may be factored by 0.25 if off-peak working is a practical option to reflect the reduced volume of traffic.

E.8 Cost Consequence Factor

E8.1 The estimated costs should be identified as:

- design costs;
- works costs including access and traffic management;
- traffic delay costs.

E8.2 The sum of the estimated costs should be used to calculate the consequence factor, determined directly from the cost by the equation:

$$(\log_n(Cost/£25{,}000/\log_n 2)) + 1$$

UNIT RATES FOR COST ESTIMATE

Activity	Unit rate	Unit	Works rate
Access costs			
Scaffolding for repair *(for 35m deck width)*	£75	per day	
Mobile elevated platform	£300	per day	
Under bridge unit	£750	per day	
Mobile Scaffold and Rail Protection Staff	£1,000	per day	
Traffic Management			
2 Lanes closed in contraflow	£1,400	per day	
1 Lane closed	£300	per day	
Traffic light control	£900	per day	
Joint Replacement			
Asphaltic	£120	per m	14 m/day
Buried	£75	per m	17 m/day
Elastomeric	£575	per m	7 m/day
Comb	£2,500	per m	3 m/day
Other or unknown	£200	per m	11 m/day
Repair			
Discrete anode CP per m width of joint	£360	per m	6 m/day
Control and monitoring equipment.			
(assumes one control cabinet per 4 joints)	£6,000	Dual Carriageway	
	£4,000	Single Carriageway	

DAILY TRAFFIC DELAY COSTS FOR DECK HINGE REPAIR

AADT (1000)	M4	M3	M2	D2	SW	SN	SL	SA
TRMM. Table Ref	5	17	32	38	41	42	42	42/2
2								£140
5						£280	£280	
6						£350	£350	
7						£430	£430	
8						£510		
10					£250	£690		
12					£320	£1,360		
14					£390			
16					£460			
18					£530			
20			£6,100	£7,200	£610			
30			£11,000	£20,000				
40		£9,200	£36,000	£62,000				
50		£13,100	£90,000					
60	£13,000	£17,000	£112,000					
80		£148,000						
100	£23,000	£194,000						
120	£57,000	£308,000						
140	£275,000	£532,000						

Note: Costs at 1998 prices.
See Table 1 for key to road codes.

EXAMPLE PROFORMA FOR QUALITATIVE RISK ASSESSMENT

Structure Key	555	**Structure Name**	Penny Brampton
Area Reference	16	**Maintaining Agent**	

Ref	Median Factor	Factor Adjustments	Likelihood (A)	Significance (B)	AxB
PROBABILITY OF FAILURE					
P1	Joint configuration and Access Type A 7 Type B 5 Type C 3 Type D 2	**Access** Difficult +2 Difficult and Moderate +1 Moderate 0	5	2	10
P2	Current Capacity at Joint 5	Current capacity < ½ Design capacity +4 Current capacity < Design capacity +2 Current capacity = Not known 0 Current capacity = Design capacity -2 Current Capacity > Design Capacity -4	7	4	28
P3	Current Condition Poor 7 Fair 5 Good 3	**Particular Defects** / **Repairs** Yes +2 / Specific Defects +2 Yes +1 / Cosmetic/no repairs 0 Structural repairs -2	3	4	12
P4	Rate of Deterioration 5	**Type of Road Joint** Elastomeric -2 Buried joints -1 All other joint 0 Open joint +3	4	2	8
P5	Future Loading 5	**Route Carried** Motorway +2 Dual A P Trunk Road +1 Single Carriageway Trunk Road 0 Lane/Local Road -2 Access Road/Footway -4	9	1	9
Average Relative Probability of Failure, P = Σ (A x B) / 13 = 67/ 13					**5.2**

Estimated Works Costs	Estimated Traffic Delay Costs	GRAND TOTAL COSTS
£65,964	£515,014	£580,978
Consequence Factor, C = ($\log_n$ (Cost / £25,000) / $\log_n$ 2) + 1		**5.5**

PROFORMA FOR QUALITATIVE RISK ASSESSMENT

Structure Key		**Structure Name**	
Area Reference		**Maintaining Agent**	

Ref	Median Factor	Factor Adjustments	Likelihood (A)	Significance (B)	AxB
PROBABILITY OF FAILURE					
P1	Joint configuration and Access Type A 7 Type B 5 Type C 3 Type D 2	**Access** Difficult +2 Difficult and Moderate +1 Moderate 0			
P2	Current Capacity at Joint 5	Current capacity < ½ Design capacity +4 Current capacity < Design capacity +2 Current capacity = Not known 0 Current capacity = Design capacity -2 Current Capacity > Design Capacity -4			
P3	Current Condition Poor 7 Fair 5 Good 3	**Particular Defects** / **Repairs** Yes +2 / Specific Defects +2 Yes +1 / Cosmetic/no repairs 0 Structural repairs -2			
P4	Rate of Deterioration 5	**Type of Road Joint** Elastomeric -2 Buried joints -1 All other joint 0 Open joint +3			
P5	Future Loading 5	**Route Carried** Motorway +2 Dual A P Trunk Road +1 Single Carriageway Trunk Road 0 Lane/Local Road -2 Access Road/Footway -4			
Average Relative Probability of Failure, P = Σ (A x B) / 13 =					

estimated works costs	Estimated Traffic Delay Costs	GRAND TOTAL COSTS
£	£	£

Consequence Factor, C = ($\log_n$ (Cost / £25,000) / $\log_n$ 2) + 1

ANNEX F NRA MANAGEMENT STRATEGY

F.1 MANAGEMENT STRATEGY : STAGE 1

Introduction

F1.1 Should a structure be found to contain a hinge joint, it shall be subject to visual inspection, and be prioritised on the basis of their external condition, as outlined in Annex C. This document sets out a Management Strategy for all structures of this type, and is indicated in flow chart format in Annex A

Data Validation

F1.2 Hinge deck structures which have not had a special inspection of the hinges should be inspected as soon as possible.

Initial Visual Inspection

F1.3 Initial inspection based on a visual examination of the hinge joints should determine the severity and location of any defects such as cracking, leaching, or corrosion products leaking through the throat, and any concrete delamination and spalling in the vicinity of the hinge. Any deformation at the hinge or sag in the deck should be recorded and measured. The investigation should determine whether the hinge throat is cracked, and if so, crack widths should be estimated. Absence of visual evidence of defects should not be taken as assurance that no defects are present, as site investigations have shown the presence of corrosion damage without external indications. Access requirements for a more thorough or Detailed Inspection should be determined at this stage. The results of this inspection will be used to assign an initial priority to the structure.

Detailed Inspection

F1.4 The objective of a Detailed Inspection is to provide more detailed information on the condition of the hinge and to quantify any defects identified during the Initial Visual Inspection. The information provided should be in a form which can be used by the assessing engineer to determine the adequacy of the structure. Close access to the hinges will be required, so access platforms, traffic management, etc., will need to be arranged. Where cracking is present, widths should be determined. This can be done from below using a selection of feeler rods of different widths at a number of locations along the throat, and these locations marked for future reference. The number of measurements taken should be carefully interpreted and averaged to ensure that a reliable value for the crack width is reported. Bridge temperature should also be recorded since crack width may be influenced by seasonal temperature variation. Any deformation of the hinge or sag in the deck should be measured. The location of any damage or corrosion products in the vicinity of the hinge, or evidence of seepage through the deck, should be noted. Whilst carrying out a Detailed Inspection, the opportunity should be taken to install monitoring pips across existing cracks to enable periodic monitoring of future changes in crack width.

F1.5 In many cases, it will be effective to combine the Initial Visual Inspection and the Detailed Inspection depending on access requirements and what is currently known about the condition of the bridge from previous inspections.

Prioritisation

F1.6 Hinge deck structures should initially be prioritised on the basis of visual condition, according to the initial prioritisation methodology outlined in Annex C. Although it is acknowledged that there are some shortcomings in this approach, it does provide a useful way of allocating resources early in the management programme. The prioritisation may be modified as further information is obtained.

When allocating priorities, consideration should be given to current and future road improvement or widening schemes and the effects these might have on the service requirements of the bridge. When sufficient information is obtained a qualitative risk assessment should be carried out as described in Annex E.

Reporting

F1.7 Agents should report all inspection information as soon as possible to the National Roads Authority, with recommendations for further investigation if necessary. This might include the use of non-destructive testing (NDT), or more extensive intrusive investigation of the hinge in order to determine the condition of the hinge reinforcement.

F1.8 Where the hinges have significantly cracked (crack widths >2mm), or where there is evidence of significant seepage through the hinge, or where deformation of the joint is evident (shear deformation across the hinge, excessive rotation at the hinge), the opportunity should be taken to determine the condition of the throat reinforcement through more detailed site investigations. One method of doing this without significant intrusion and de-stressing of the reinforcement is to carefully drill small holes through the cracked hinge throat, and inspect bars using a borescope. If there is significant seepage, limited concrete condition testing should be carried out at the hinge to supplement data from earlier principal or special inspections. This should include measurement of chloride content, cement content, half-cell potential, and any other measurements deemed necessary by the engineer. Particular attention should be given to the condition of the vertical link reinforcement around the hinge, since this contributes significantly to the integrity of the joint. It may also be possible to identify the type of reinforcement used and whether it is misplaced.

F1.9 Where there are no indications of significant cracking, seepage, or other defects, no further immediate action is required provided the capacity of the hinge, assessed as described in F1.10 et seq, is adequate. Normal inspection and maintenance arrangements will apply. Agents should ensure that information relating to the condition of the hinge is included and recorded for use in the structural assessment. Where significant cracks have been observed, or where other defects or signs of deterioration are present which give cause for concern the use of periodic monitoring should be considered: see F1.20.

Structural Assessment

F1.10 A Preliminary Assessment should be carried out initially to determine the adequacy of the hinge. This should be based on a realistic analysis of the bridge deck. The hinges can be modelled as pins. Skews less than 10° will only have a small effect and can be ignored. Skews greater than 10° should be modelled correctly, with the line of the hinges positioned as it occurs in the deck. The analysis should be used to assess all load effects in the deck in accordance with NRA BD 44. The shear capacity of the hinge is just one of the effects that should be considered: shear in the cantilever section can be the limiting factor in some deck configurations.

F1.11 Simplified conservative methods can be used to determine the shear capacity of the hinge and guidelines for an appropriate methodology are presented in main text of the Advice Note. Due account should be taken of the condition of the hinge determined through NDT and invasive inspection (see F1.15 et seq). If the adequacy of the joint cannot be demonstrated then a Detailed Assessment should be carried out. A methodology is presented in the main text of the Advice Note. Alternative methods can be used provided it is demonstrated that the hinge behaviour is adequately modelled.

F1.12 Particular attention should be paid to the method of analysis adopted, and whether the assumptions made about the condition of the hinge are appropriate. The validity of any departures from standard previously granted should also be confirmed. It is recognised that previous assessments concentrated on the effects of the 40t assessment live load, and it may be necessary to reassess the structure in its present (i.e. deteriorated and cracked) condition, taking account of construction defects such as poor concrete compaction, curing and reinforcement misalignment, where known.

F1.13 If the assessment reveals no strength deficiency and there are no concerns about the serviceability behaviour and durability of the structure, no further action need be taken and the bridge can be returned to the normal management cycle. However, because of the vulnerability of deck hinges to deterioration, Agents should review existing structural assessment reports as part of the NRA Eirspan Bridge Management System, and carry out new assessments as appropriate: see F3.1.

F1.14 Where assessment has shown that the capacity of the hinge is not adequate then action needs to be taken to ensure the safety of the structure and the road users. Depending on the condition of the structure, the rate of deterioration, and the conclusions of the strength assessment, consideration should be given to periodic monitoring, interim measures and the development of remedial works leading to a long-term solution, as outlined in the following sections. An important aspect of identifying the most appropriate action is the management of risk: see F1.24.

Invasive Inspection and Non-destructive Testing

F1.15 Detailed assessment requires accurate information on the condition of the hinge and its geometry, and this can only be obtained by invasive inspection, testing and non-destructive methods. Full advantage should be taken of NDT techniques, but if it is considered that there is still insufficient information about the condition of the hinge and its reinforcement for assessment purposes, further invasive testing to expose the reinforcement in the hinge may be necessary. Such investigations will be subject to technical acceptance procedures and should be supported by a full technical appraisal – see F1.17. This should safeguard the structure during the course of the work, set down the type of investigation proposed and details of the expected outputs, and outline how the results of the investigations will be used.

F1.16 Consideration should be given to selecting the most appropriate test methods and the most suitable test location(s) on the bridge using the risk assessment approach outlined in F1.24. Consideration should also be given to, drainage paths and the severity of defects, together with safety, access and traffic management issues. Where invasive testing involves de-stressing the hinge joint, the additional loading carried by adjacent sets of hinge bars should be assessed. Analysis should be used to determine whether it is necessary to prop the bridge when individual hinges are broken out. If propping is to be utilised, the analysis should be extended to determine the magnitude and position of the propping load, its effect on the hinge, and its effect on elements remote from the hinge.

F1.17 Agents should submit detailed proposals for invasive testing to the National Roads Authority for discussion and agreement, including the method, timescale, cost, materials tests and inspection, reinstatement procedures, traffic management, noise control and contingency measures, etc. Particular attention should be given to planning the reinstatement of the test areas, the selection of materials, and method of reinstatement, taking account of time constraints, potential weather conditions and engineering requirements. Contingency measures should be planned to take into account difficulties encountered during the invasive testing process, including the condition of the exposed hinge reinforcement, unexpected delays and weather conditions.

F1.18 NDT methods such as impact echo, radiography, acoustic emission, and thermography, etc, may be used to minimise the need for invasive inspection. Radiography and acoustic emission have been used on hinge decks in the UK, and appear to be useful techniques. In the case of acoustic emission some fieldwork and research has been undertaken in the UK to assess whether the technique will detect cracking in concrete or reinforcement corrosion. Not all of this work has been related to hinge structures. Further research has been carried out in the UK to characterise the outputs, in order to build confidence that the technique can be used to quickly gauge conditions on site. Whilst NDT methods at present are unlikely to give definitive indications of defects, they may be used to assist determination of the variations in condition along joints, and also allow coverage of large areas in a relatively short time. The results, properly interpreted and compared to known conditions at one or more locations derived by invasive inspection, should give a good indication of conditions, or point to where further invasive inspection may be necessary. In addition to the above test methods, reinforced concrete tomography using a gamma ray source does offer some promise as a means to detect defects and corrosion in hinges, though it is yet to be fully calibrated against known defects in deck hinges or trialled in the UK.

F1.19 Recent experience of invasive inspection and testing of hinge deck structures has been gained in South Wales. This was undertaken using water-jetting to create small ‘pockets’ for inspection. These investigations revealed that:

i) As-built drawings are sometimes inaccurate, and there can be construction defects that significantly affect an assessment, such as: displaced joint formwork; concrete bridging across the gap; misaligned throat reinforcement that does not cross the throat at the same location.

ii) Leakage is not necessarily a true indicator of corrosion potential. Severe pitting corrosion can exist within outwardly ‘dry’ joints, where the carriageway joint filler soaks up chlorides in small but concentrated quantities. On the other hand severe leakage may dilute chlorides and wash out the joints. Even with full exposure, pitting corrosion is not easy to see and measure, and it is possible that the use of a borescope alone may miss important defects.

Monitoring

F1.20 Where significant defects are present and there are some concerns about the serviceability and strength of the hinge, a regime of periodic monitoring and inspection may provide a short-term solution pending implementation of a permanent solution such as repair or strengthening of the hinge, or replacement of the bridge. Some general guidelines on monitoring are presented in NRA BD 79 ‘Management of sub-standard road structures’. Monitoring can be considered in cases where bridge works are likely to cause excessive disruption to traffic or where costs would be disproportionate to the benefits achieved. The objective of the monitoring is to enable the structure to remain in service, while examining its behaviour to identify any changes indicating a reduced reliability. This might include monitoring some or all of the following:

i) progressive horizontal and vertical movement at the joint;

ii) changes in crack width and length;

iii) movement under traffic loading;

i) increasing strains at critical locations;

ii) ongoing material deterioration.

F1.21 The sophistication of the monitoring scheme will depend on the particular circumstances (condition, assessed capacity, access, conclusions of risk analysis, etc.). At the lowest level, monitoring can be based on a visual approach to avoid the need for sophisticated instrumentation or logging systems. In some cases it may be appropriate to utilise continuous monitoring using strain or other movement gauges. The intervals for monitoring should be appropriate for the structure (e.g. 3 months to 1 year), depending on the nature and severity of the deterioration, and the potential risk to the network.

F1.22 Monitoring of movement at the hinge can be undertaken using a demountable strain gauge to take manual measurement between DEMEC pips bonded either side of the hinge. Displacement transducers can be mounted across the hinge if continuous measurements are required. The location of the instrumentation on the bridge depends on ease of access but would normally be on the underside of the deck. Manual monitoring is perhaps best used as part of an initial investigation into structural performance. Automatic or remote monitoring can be used to enable prior warning of structural problems.

F1.23 The use of monitoring to ensure the safety of a structure requires careful management if it is to be used effectively and reliably. The on-going management of a monitoring scheme is considered in F3.5. Agents should discuss and agree the proposals with the National Roads Authority.

Risk Management

F1.24 Risks should be assessed considering usage factors (defined as the ratio of the ultimate load effect to the assessed hinge capacity: see Annex F), rates of deterioration, potential modes of failure, and network factors such as traffic volume and Heavy Goods Vehicle (HGV) loading over and under the bridge, bridge location and alternative routes, etc. An initial prioritisation can be carried out using the methodology outlined in Annex C. Where a more detailed qualitative assessment is required Annex E can be used once more detailed information about condition and capacity are available. Example and blank proformas for the qualitative risk assessment are given, together with detailed guidance on the methodology. Structures with hinges with a likelihood factor of 6 or higher determined according to the Risk Assessment in Annex E, are likely to require management effort in the near future to ensure they will not become substandard. The higher the likelihood factor the more urgent the need for remedial action is likely to be.

Interim Measures

F1.25 If the results of these investigations reveal an unacceptable risk to the integrity of the structure, Interim Measures should be developed and implemented to safeguard the road network. Such measures might include temporary propping, load reduction (lane or weight restrictions), temporary repair or strengthening of the hinge, or some combination of these. The procedures outlined in NRA BD 79 'Management of sub- standard road structures' should be instigated, and Technical Acceptance procedures for the temporary works will apply. Interim Measures are by definition temporary for a specified period of time, pending the formulation and execution of a permanent solution (permanent repair, strengthening or replacement). Careful management is required to ensure that Interim Measures are installed correctly and remain effective for as long as they remain in service : see section F3.3.

Data Management

F1.26 As part of this Maintenance Strategy, the NRA Eirspan Bridge Management System will be used to manage the programme of work on hinge deck structures. To assist the National Roads Authority in developing an effective Management Strategy, it is important that Agents input the required data. This will assist Agents in managing their own programmes as well as allowing the Agency to produce and assess national progress reports. It will be necessary for Agents to populate the NRA Eirspan Bridge Management System with the necessary 'historical' data for work already completed.

Research

F1.27 Research was commissioned at Transport Research Laboratory (TRL) in the UK to investigate the durability of deck hinges in a deteriorated condition. The research was based on a programme of laboratory testing, supported by a theoretical study to model the behaviour of hinge deck structures. This, along with previous research, provided the basis for this Management Strategy. Further research is being carried out to assist the development of robust and reliable NDT methods which can be used to determine the condition of the steel reinforcement at and adjacent to the hinge throat. Advantage should be taken of these developments where appropriate.

F.2 MAINTENANCE AND REMEDIAL WORKS

Maintenance

F2.1 For all hinge deck bridges, high priority should be given to preventing further deterioration of hinge joints by maintaining drainage in working order and the integrity of deck waterproofing and expansion joints, including pipe bays where appropriate. Bids for remedial works should be classified as essential maintenance. Advantage should also be taken during any planned re-waterproofing or resurfacing work to undertake a detailed visual inspection of the concrete and reinforcement at the hinges.

Remedial Works

F2.2 Irrespective of the interim measures adopted, proposals should be devised to provide for the long-term safety/stability of the structure. Remedial options will vary between minor concrete repairs, reconstruction of the hinges, safeguarding measures such as props, consideration of re-articulation of the deck and complete reconstruction. All maintenance operations measures are likely to affect the integrity of the structure and will therefore be subject to Technical Acceptance procedures. Where they are significant then independent checks will also be required, and they will be considered as Category 3.

F2.3 The repair of hinges is made particularly difficult due to poor access, congested reinforcement and traffic management issues. Advice is given below on possible repair methods.

F2.4 Concrete replacement is an option for repairing deteriorated concrete. Information on concrete replacement is provided in NRA BD 27 'The Protection and Repair of Concrete Road Structures'. The HA/CSS/TRL publication, 'Best practice guidance for concrete repair' details current thinking on best practice to be adopted for concrete repair. Unless such practices are adopted, it is likely that concrete repairs will only be partially effective in minimising future corrosion of reinforced concrete.

F2.5 Other options such as the use of electrochemical techniques, including cathodic protection may be feasible in the future, in some circumstances, as a means to minimise future corrosion. However, electrochemical techniques are currently not advised where there are sensitive structural details such as hinges (refer NRA BA 83 'Cathodic protection for use in reinforced concrete structures'), and would need very careful assessment and specialist advice before adoption.

F2.6 There are a number of alternative commercial repair systems available to manage deteriorating reinforced concrete such as, chloride extraction, cathodic protection, and active moisture reduction systems. The effectiveness of these particular remedial methods for use on hinge joints is not yet proven and as such they are not considered appropriate at this time.

F2.7 Where a hinge joint has deteriorated so badly that it is practically or economically beyond repair, for example where the reinforcement is so badly corroded that it cannot be satisfactorily reinstated, then it may be necessary to replace the whole element.

F.3 MANAGEMENT STRATEGY : STAGE 2

Future Management

F3.1 Notwithstanding the conclusions of the appraisal outlined in the preceding sections of this document, the vulnerability of concrete hinges to deterioration requires that bridges containing these details need careful management in the long-term. The paragraphs below define how the structure is to be managed in the future depending on the outcome of this assessment.

F3.2 Where assessment has shown that the hinges are performing satisfactorily and their strength is adequate, no special action is required other than the normal inspection/maintenance activities. Experimental tests and theoretical analysis of hinge joints in as-built condition and with simulated defects have shown that there is a generous factor of safety in capacity over applied loading. However, it is likely that capacity may be reduced in structures exhibiting significant deterioration. Where hinge deck structures are exhibiting significant deterioration, Agents should review existing structural assessment reports as part of the NRA Eirspan Bridge Management System, and carry out new assessments as appropriate. This may be triggered by changes in condition as reported through the routine Principal Inspection regime.

Management of Interim Measures)

F3.3 Where Interim Measures have been adopted, Agents should ensure that they remain effective throughout their service life. In devising and planning the Interim Measures, consideration should be given to how they are to be managed, and procedures for their inspection and maintenance need to be developed and incorporated into the routine Principal Inspection regime.

F3.4 Interim Measures are not a permanent solution and should only be continued while an appropriate long-term solution is determined and implemented. The service life of these measures should be specified at the time of implementation, and should take due account of the condition of the hinge, the rate of deterioration, the assessment results, and the risk analysis. All Interim Measures should be subjected to a 2-yearly review at which time the decision should be taken whether to continue with them, or whether a permanent solution is required. The review should take account of the current condition, in-service behaviour, traffic levels, and the costs and benefits of implementing a permanent solution.

Management of Monitoring

F3.5 The purpose of monitoring is to identify changes in behaviour which might compromise the safety of the structure. Effective management requires that the monitoring be focussed on critical measurable parameters (e.g., condition, crack widths deflections, sag, strain) and that acceptable thresholds are defined. If any of these thresholds are exceeded, the hinge should be examined urgently to determine the next course of action. The monitoring should be continued until an appropriate long-term solution is determined and implemented. The monitoring period should be specified at the time of implementation and should be subject to regular review, taking due account of the changing condition and behaviour of the structure. General advice on the formulation of monitoring systems is given NRA BD 79, 'Management of sub-standard road structures'.

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