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The Assessment of Reinforced Concrete Half-Joints

AM-STR-06007
June 2014

AM Asset Management & Maintenance

Standards

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Volume 3 Section 4
Part 4
NRA BA 39/14

The Assessment of Reinforced Concrete Half-Joints

June 2014

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Summary:

This Advice Note provides criteria for the assessment of reinforced concrete half-joints.

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VOLUME 3 ROAD STRUCTURES: INSPECTION AND MAINTENANCE

SECTION 4 ASSESSMENT

PART 4

NRA BA 39/14

THE ASSESSMENT OF REINFORCED CONCRETE HALF-JOINTS

Contents

Chapter

1. INTRODUCTION

General

1.1 This Advice Note gives guidance on the assessment of concrete half-joints at both the ultimate and serviceability limit states. It is based on the findings of a research project[1] which investigated the behaviour of a range of half-joints, combining a variety of reinforcement layouts with different types of bearings.

1.2 The main problem in assessing the long-term durability of half-joints is the difficulty in determining the relevant strains and crack widths. This Advice Note presents an elastic analysis for doing this which has been modified to allow for the non-linear behaviour in the region of the re-entrant corner. An explanation of the analysis is given in Appendix A together with a worked example of the method in Appendix B.

1.3 The arrangement and detailing of the reinforcement is an important factor in the performance of a half-joint. The superseded design requirements given in BS 5400: Part 4: 1990[2] (hereinafter referred to as Part 4) are mainly concerned with the ultimate limit state. As a result of the research by Clark[1] a need for additional reinforcement to ensure a satisfactory performance under service loading has been identified. When assessing an existing half-joint, it is therefore necessary to compare the reinforcement provided with the combined recommendations of the Advice Note and NRA BD 44, the Assessment of Concrete Road Bridges and Structures.

1.4 Any reference in this Advice Note to a British Standard is to that Standard as implemented by the appropriate Departmental Standard.

Scope

1.5 The advice given in this document is applicable to both upper and lower reinforced concrete half-joints. It may also be applied to pre-tensioned and post-tensioned prestressed half- joints which for this purpose can be considered as reinforced concrete elements. For pre-tensioned members, the prestressing force and tendons should be ignored, but for post-tensioned members the prestressing force should be considered as an external force acting on the half-joint.

1.6 The behaviour of half-joints has been found to be influenced by the eccentricity of the reaction and the type of bearing. This Advice Note covers the use of both rigid and flexible bearings.

1.7 The criteria given in the Advice Note are intended to be used in conjunction with that given in NRA BD 44. The Advice Note covers both serviceability and ultimate limit states.

Implementation

1.8 This Advice Note should be used in all assessments of reinforced concrete half-joints.

2. SERVICEABILITY LIMIT STATE

General

2.1 Although assessments are normally carried out at the ultimate limit state, for half-joints, because of durability considerations, it is advisable to check the serviceability limit state in an assessment. The serviceability criteria stipulated in Part 4 for design should, in general, be adopted for the assessment of half- joints. However, the National Roads Authority may, under particular conditions of exposure, feel it necessary to modify the Part 4 requirements. Methods for determining strains and crack widths are given in this Advice Note.

2.2 Where the serviceability criteria are exceeded, inspection of the half-joint should be undertaken to confirm the condition of the joint. If there is no cracking and the load carrying capacity of the joint is adequate, more frequent future inspection may be the only course of action to be adopted. Where there is extensive cracking or spalling, it is unlikely to be practicable to strengthen the joint just to satisfy the serviceability criteria. Repair of damaged concrete and reinforcement may also prove difficult where access is restricted. Therefore it is important that such joints are regularly inspected to monitor their performance and measures are taken to prevent water reaching the repaired sections.

2.3 Tests on half-joints have shown that cracking is generally initiated at the re-entrant corner as a result of shrinkage, and that maximum crack widths subsequently occur at this point. The overall cracking pattern which develops is dependent on the reinforcement arrangement, the eccentricity of the reaction and the type of bearing. However, for half-joints which have been adequately designed for shear, it can be assumed for the purpose of analysis, that cracking will be concentrated in a zone which extends at 45° from the re-entrant corner towards the top of the section, as shown in Figure 2.1.

Strains

2.4 The strain distribution assumed in a half-joint at the serviceability limit state is illustrated in Figure 2.2. It can be seen that the maximum concrete tensile strain occurs at the re-entrant corner B and the maximum compressive strain E_c occurs at the top surface of the member at A vertically above the line of intersection of the neutral axis with a line at 45° from the re-entrant corner. However the strain distribution in the tensile zone is non-linear and this leads to an under-estimate of the tensile strain at the extreme fibre of the concrete at the re-entrant corner when using a linear elastic analysis. The non-linearity in the strain distribution is as a result of slip occurring between the reinforcement and the concrete. Therefore, whilst the compressive strain in the concrete and the tensile strain in the reinforcement can be determined directly from an elastic analysis, as shown in Appendix A, some modification of the extreme fibre concrete tensile strain is required. A factor K_1, derived from the tests on half-joints, is applied to the strain at the re-entrant corner, $\in_1$, determined by an elastic analysis, to give:

$$\in' = K_1 \in_1 \qquad \text{eq. (i)}$$

where $\in'$ is the modified re-entrant corner strain and K_1 is taken as 2.3 when inclined reinforcement is present and 3.5 where inclined bars are omitted.

2.5 In addition to slippage some allowance must be made for the tension stiffening effect in the cracked concrete section. The complete expression for $\in'$ therefore becomes:

$$\in' = K_1 \in_1 - K_2 bhf_t / E_s \in_s A_s \gamma_m \qquad \text{eq. (ii)}$$

where

K_2 is derived from test evidence and can be taken as 0.3×10^{-3}

b is the breadth of the half-joint in mm.

h is the depth of the half-joint in mm.

f_t is the modulus of rupture of the concrete, taken as $0.556\sqrt{f_{cu}}$, in N/mm².

E_s is the elastic modulus of steel in N/mm².

$\in_s$, is the strain at the steel level in the direction normal to a 45° line from the re-entrant corner, determined from an elastic analysis. (See Figure 2.2).

f_{cu} is the characteristic concrete cube strength, in N/mm².

γ_m is the appropriate partial safety factor for material strength.

A_s is the effective area of steel in mm² normal to a 45° crack from the re-entrant corner, determined from:

$$A_s = \sum_{i=1}^{\eta} A_{si} \cos^2 (45-\beta_i),$$

where

A_{si} is the area of one layer of reinforcement at an angle β_i to the horizontal.

Crack Width

2.6 The predicted maximum crack width, w, at the re-entrant corner is taken as the lesser of the values from the following equations using the modified strain ϵ' from equation (ii):

$$w = \sqrt{2}(a - 0.5y) \in' \qquad \text{eq. (iii)}$$

and

$$w = 3a_{cr} \in' \qquad \text{eq. (iv)}$$

where

a is the distance of the vertical reaction taken at the front edge of a rigid bearing or centre line of a flexible bearing from the re-entrant corner in mm.

y is the dimension of the fillet in mm.

a_{cr} is the distance from the nearest bar to the point where the crack width is calculated in mm.

2.7 Equation (iii) is based on the average strain over a distance of $\sqrt{2}(a - 0.5y)$ on either side of the crack, as illustrated in Figure 2.3. The crack width determined by this equation can become unrealistic for large values of a, since in such cases more than one crack will occur and the maximum crack width reduces. In these situations, the maximum crack width would be more accurately determined by assuming the element behaves as a long cantilever and using equation (iv) which is based on equation 26 of Part 4. For the purpose of this Advice Note, equation (iv) will govern when a is greater than $3\,a_{cr}/\sqrt{2} + 0.5y$. The predicted maximum crack width should be checked against the limit stipulated in Part 4.

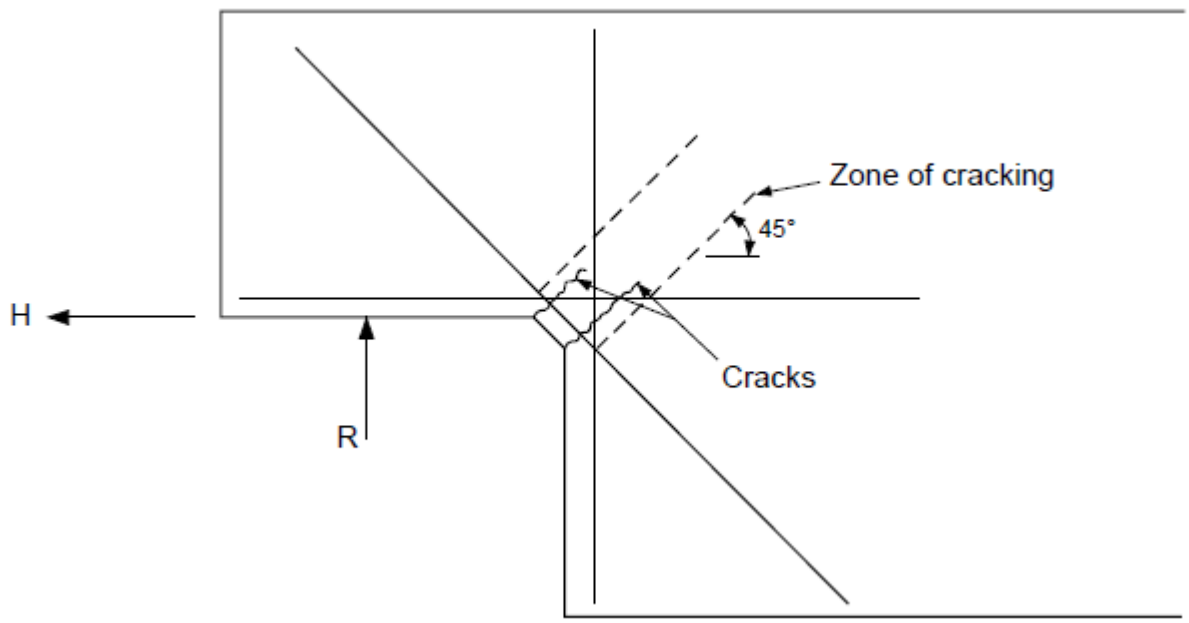

Figure 2.1 – Zone of cracking

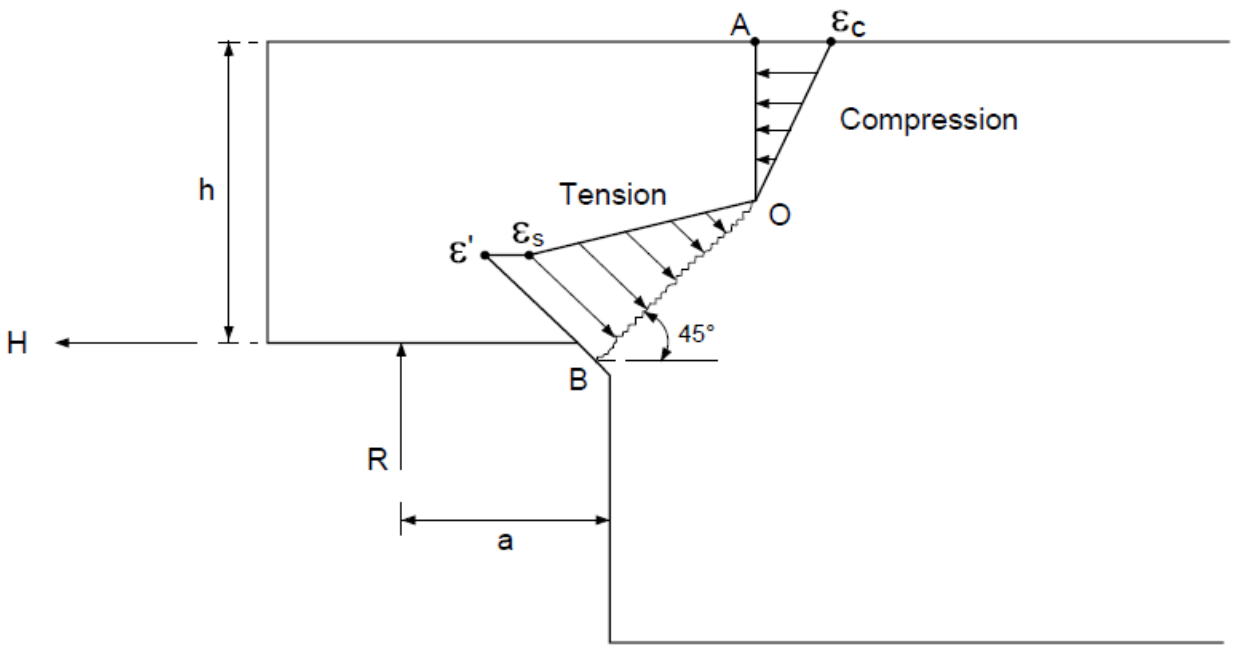

Figure 2.2 – Strain distribution

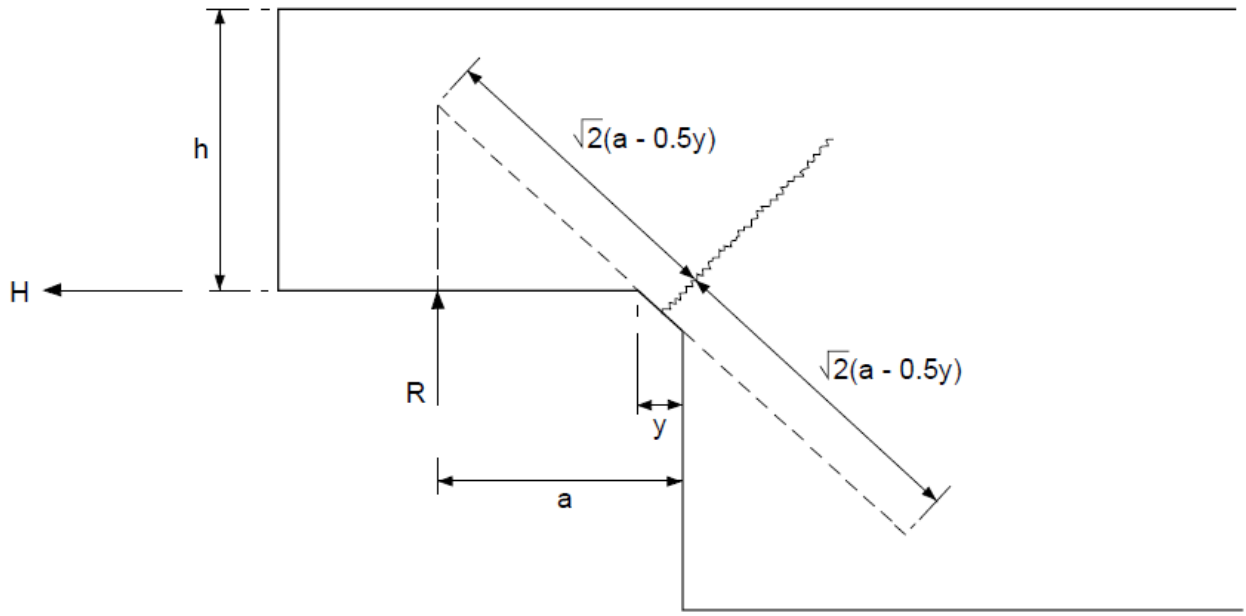

Figure 2.3 – Crack width model

3. ULTIMATE LIMIT STATE

General

3.1 The strength of a half-joint for assessment of existing structures should be determined in accordance with the requirements of NRA BD 44.

Horizontal Forces

3.2 When determining the horizontal forces to be resisted by the reduced section of a half-joint, as shown in Figure 7 of NRA BD 44, consideration should be given to the additional horizontal forces that may occur at the bearing. Clause 7.2.3.4 of NRA BD 44 lists come of the possible causes of these forces. If significant, such forces can reduce the load carrying capacity of the section by causing premature cracking over the bearing.

3.3 In the tests on half-joints where such cracks have occurred, the cause was mainly attributed to high rotation of the bearing resulting in a concentration of loading at one end. Extensive cracking was observed during tests on half-joints with soft rubber bearings. Therefore their use is not recommended for half-joints. It is important to check that the type of bearing used for half-joints is capable of accommodating the rotation at the support.

3.4 In assessing half-joints which do not contain sufficient reinforcement to resist these horizontal forces, a tensile strength of $0.566\sqrt{f_{cu}}/\gamma_m$ for concrete may be assumed.

3.5 For wide slabs with a number of bearing positions across the deck, consideration should be given to the lateral load distribution. Figure 3.1 illustrates a strut and tie system representing the load distribution as viewed in elevation at the end of the slab. The lateral reinforcement in the non-loaded face can be assumed to be subject to a tensile force T where,

$$T = P(b - b_w)/4d$$

and

P is the vertical reaction at each bearing at the ultimate limit state in kN

d is the effective depth to the lateral reinforcement in mm

b is the effective width of the slab in mm, taken as the lesser of the spacing of the bearings in the transverse direction or the width of the bearing plus 2d.

b_w is the width of the bearing in mm.

3.6 The lateral reinforcement within the length of the reduced depth section can be considered to contribute to resist T.

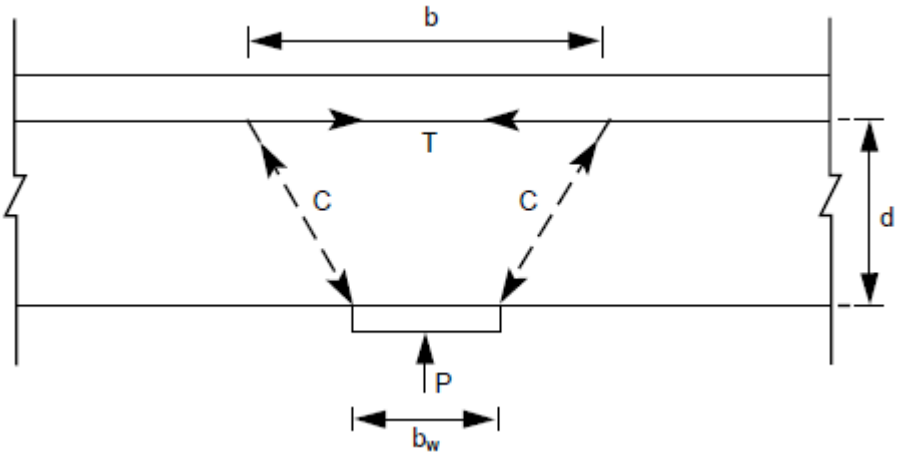

Figure 3.1 – Lateral Load distribution

4. REINFORCEMENT

General

4.1 The arrangement of reinforcement in a half-joint contributes significantly to the behaviour of the element, particularly under service loads. Whilst certain combinations of horizontal, vertical and inclined bars will be satisfactory in ensuring an adequate ultimate load capacity, the resultant strains and crack widths under service loads may be unacceptable. It is therefore important to satisfy the necessary criteria for both serviceability and ultimate conditions to ensure the long term durability of these joints.

4.2 Where there is evidence of corrosion of reinforcement in a half-joint, allowance should be made for any loss of cross-section in assessing the strength of the element. In addition, consideration should be given to the effects due to corrosion on the fatigue life of the reinforcement and an assessment in accordance with NRA BD 38 may be required.

Inclined Links

4.3 The presence of inclined links in a joint greatly improves the behaviour under service loading. Results from tests show that maximum crack widths are considerably greater for joints reinforced with only vertical links as compared to those reinforced with inclined links. Position of the link relative to the re-entrant corner also influences the crack width and links should therefore be positioned as accurately as possible. Strains and crack widths can be determined from the equations in paragraphs 2.4 and 2.5 of this Advice Note for all joints reinforced with inclined links, vertical links or a combination of the two.

Horizontal Reinforcement

4.4 Horizontal reinforcement in both top and bottom faces of the reduced section of a half-joint mitigate the effects of the concentrated load at the bearing and the effects of any applied loads. Horizontal bars provided at the bearing face of the joint help in resisting horizontal tensile stresses that may develop in the concrete at this position. The areas of reinforcement provided in a joint should be checked against the requirements of paragraphs 4.1 and 4.2 of this Advice Note. Ideally, a minimum area of secondary reinforcement in accordance with the superseded design standards should be present. However, where this is not the case in an existing half-joint, regular inspection should be undertaken to monitor the development of any cracks.

Other Factors

4.5 Where half-joints are part of the pre-tensioned or post-tensioned member, the requirements for the provision of reinforcement in the transmission zone or end block should be determined in accordance with NRA BD 44.

5. REFERENCES

5.1 Clark, L. A. and Thorogood, P. Serviceability Behaviour of Reinforced Concrete Half-Joints. TRRL Contractor Report 70, 1987.

5.2 BS 5400: Steel, Concrete and Composite Bridges: Part 4. Code of Practice for Design of Concrete Bridges. BSI, 1990. *[implemented by BD 24]*

5.3 NRA Design Manual for Roads and Bridges

NRA BD 44 The Assessment of Concrete Highway Bridges and Structures

NRA BD 38 Assessment of the Fatigue Life of Corroded or Damaged Reinforcing Bars

NRA BD 24 The Design of Concrete Highway Bridges and Structures. Use of BS 5400: Part 4: 1990

6. ENQUIRIES

6.1 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:

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……
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APPENDIX A - CALCULATION OF STRAINS BY ELASTIC ANALYSIS

Figure A.1 illustrates the elastic model from which ϵ_1, the extreme fibre concrete tensile strain at the re-entrant corner, can be determined for any arrangement of reinforcement. It is assumed that under service loading, a half-joint of width b has a single 45° crack from the re-entrant corner extending to point 0, above which the concrete is in compression. The principal strains are assumed to be either perpendicular to the crack or perpendicular to the line of principal compression, depending on the position in the section being considered, and are proportional to the distance from the point of zero strain, point 0.

Figure A.2 shows the free body to the left of the crack and line of principal compression for the case of one layer of reinforcement with area A_{si} at an angle β_i. If the extreme fibre compressive strain is ϵc then the strain perpendicular to the crack at the steel level is

$$\in_i = \in_c (d_i - x)\sqrt{2}/x \tag{1}$$

where d_i is the effective depth to the layer of reinforcement under consideration. The strain in the direction of the steel, resolving strains in accordance with Mohr's circle is

$$\in_{si} = \in_i \cos^2(45 - \beta_i) \tag{2a}$$

A better agreement with the test data is achieved by considering the strains as displacements and equations re-written as

$$\in_{si} = \in_i \cos(45 - \beta_i) \tag{2b}$$

The steel stress based on equation (2b) is

$$f_{si} = E_s \in_{si}$$

where E_s is the elastic modulus of steel and the steel force is

$$F_{si} = A_{si} f_{si}$$

Considering the forces acting on the free body, the horizontal component of one layer of reinforcement is

$$F_{hi} = F_{si} \cos\beta_i = A_{si} E_s \in_{si} \cos\beta_i$$

The concrete force C acting at a depth of x/3 is

$$C = E_c \in_c bx/2$$

where E_c is the elastic modulus of concrete.

Therefore for n layers of reinforcement as shown in Figure A3, horizontal equilibrium is given by

$$H + C - \sum_{i=1}^{Nn} F_{hi} = 0 \tag{3}$$

and for moment equilibrium about 0

$$R(a+h-x) + H(h-x) - C(2x/3) - \sum_{i=1}^{n} F_{si}\sqrt{2}(d_i - x)\cos(45-\beta_i) = 0 \tag{4}$$

where

x	is the depth to the neutral axis
d_i	is the depth of A_{si} at the position of the crack
H	is the horizontal reaction
R	is the vertical reaction

Equations (3) and (4) can be solved iteratively to give x and ϵ_c.

The steel strains can be determined from equation (2b) and the extreme fibre concrete tensile strain ϵ_1 at the re-entrant corner obtained from:

$$\epsilon_1 = \epsilon_c(h + 0.5y - x)\sqrt{2}/x$$

where y is the dimension of the fillet

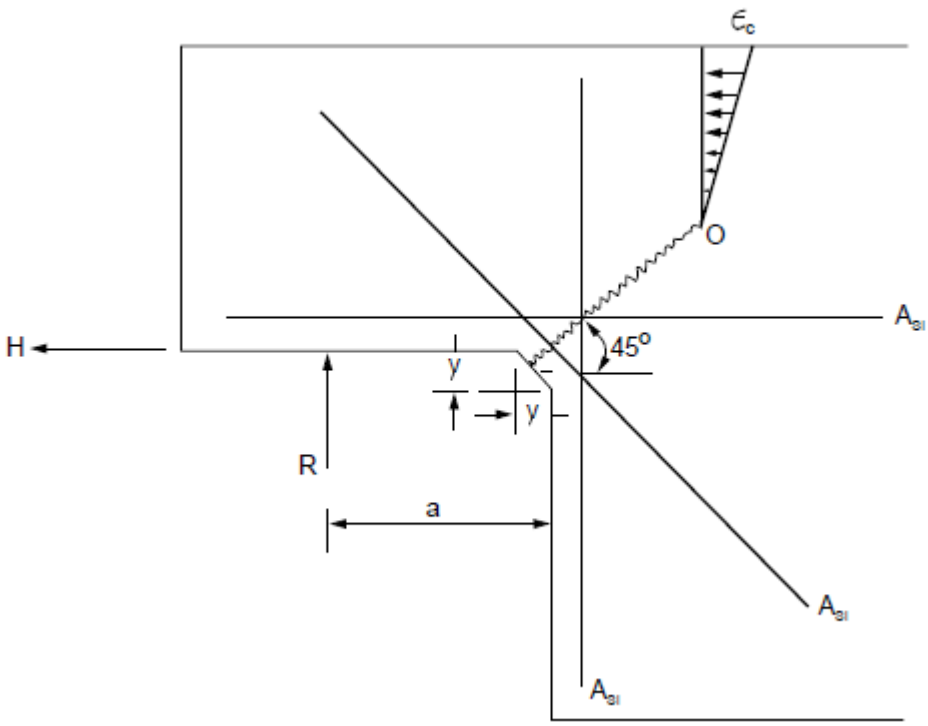


Figure A1

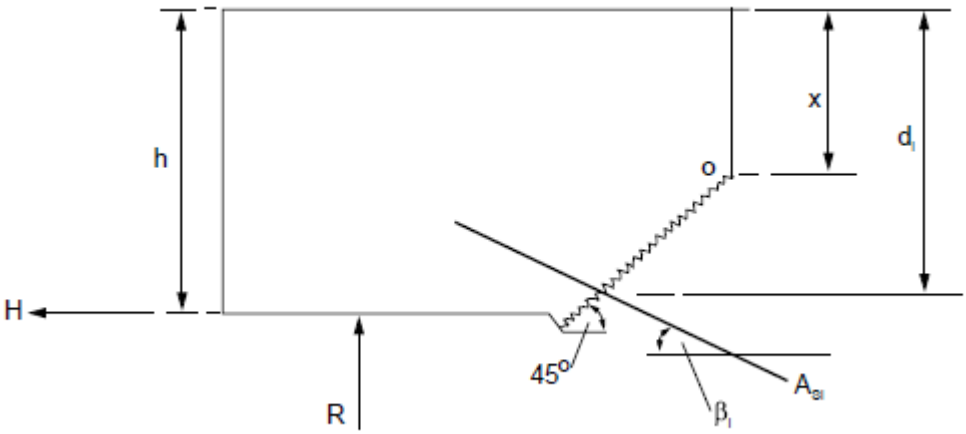


Figure A2

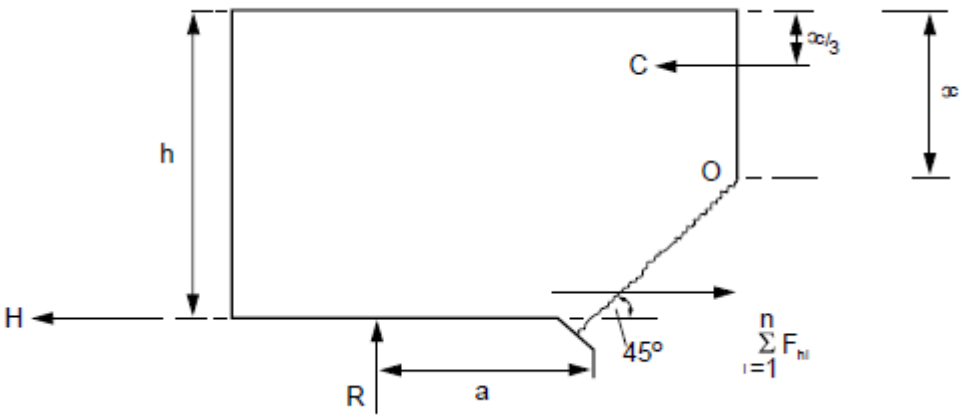


Figure A3

APPENDIX B - EXAMPLE OF CRACK WIDTH CALCULATION

Figure B1 shows the half-joint detail in a reinforced concrete voided slab which has been designed for 37.5 units of HB loading. The total width of the slab is 9.9m and the effective width of slab per bearing is taken as 1.1m. The main inclined reinforcement consists of T20 links (now referred to as 'B'20 links) in two layers giving an area per bearing in each layer of 2510mm^2. T12 U-bars and links provide the horizontal and vertical reinforcement respectively giving an area per bearing in each direction of 452mm^2.

a) Section Details

$h = 710mm$ $n = 4$
$b = 1100mm$ $l_i = (d_i - x)$
$a = 305mm$ Cover = 35mm
$y = 100mm$
$A_{s1} = 2510mm^2$, $ß_1 = 60°$
$d_1 = 710 + 50 - [(35+10)/\cos 15°] \sin 45° = 727mm$
$A_{s2} = 2510mm^2$, $ß_2 = 60°$
$d_2 = 760 - [(100+10)/\cos 15°] \sin 45° = 679mm$
$A_{s3} = 452mm^2$, $ß_3 = 0°$, $d_3 = 710 - 61 = 649mm$
$A_{s4} = 452mm^2$, $ß_4 = 90°$, $d_4 = 710 - 100 = 610mm$

$$A_s = \sum_{I=1}^{4} A_{si} \cos^2 (45 - ß_i) = 5136mm^2$$

b) Material Properties

$f_{cu} = 30N/mm^2$
$f_t = 0.556 \sqrt{f_{cu}} = 3.05N/mm^2$
$E_c = 28kN/mm^2$
$E_s = 200kN/mm^2$

c) Loading

R = 1057kN, H = 0 (From Part 4 serviceability loading).

d) Crack Width Analysis

Taking the equilibrium equations (3) and (4) in Appendix A, substitute for ΣF_{hi} and C in terms of A_{si}, ϵ_c, $ß_i$, d_i and x. This gives

$$E_c\epsilon_c\, bx/2 - \sum_{i=1}^{4} (A_{si}E_s\epsilon_c(d_i - x)\sqrt{2}\cos(45 - ß_i)\cos ß_i)/x = 0 \qquad \text{(i)}$$

and

$$R(a+h-x)-E_c\epsilon_c\, bx^2/3 - \sum_{i=1}^{4} 2.A_{si}E_s\epsilon_c(d_i-x)^2\cos^2(45-ß_i)/x = 0 \qquad \text{(ii)}$$

Evaluate $\sum_{i=1}^{4}$ ( ) term from equation (i) for n = 4

$$\therefore \quad \sum_{I=1}^{4} (\ ) = [343 \times 10^6\epsilon_c(727- x) + 343 \times 10^6\epsilon_c(679- x) + 90.4 \times 10^6\epsilon_c(649 -x)] /x$$

Substitute in equation (i) and find x

$$E_c\epsilon_c\, bx/2 - \epsilon_c \,.\, 10^6 (54.09 \times 10^4 - 776.4x)/x = 0$$

$$\therefore x = 164\text{mm}$$

Evaluate $\sum_{i=1}^{4} (\quad)$ term from equation (ii) and substitute

for x = 164mm

$$\therefore \quad \sum_{i=1}^{4} (\quad) = 356 \times 10^{10}\epsilon_c$$ Substitute in equation (ii) and find ϵ_c

$$R(a + h - 164) - E_c\epsilon_c b\, 164^2/3 - 356 \times 10^{10}\, \epsilon_c = 0$$

$$\therefore \quad \epsilon_c = 2.34 \times 10^{-4}.$$

Determine the strain at the concrete surface ϵ_1,

where $\epsilon_1 = \epsilon_c\,(h + 0.5y - x)\,\sqrt{2}\,/x$

$$\epsilon_1 = 1.2 \times 10^{-3}$$

Determine the strain at the level of the outermost reinforcement layer, n = 1
where

$$\epsilon_{s1} = \epsilon_c\,(d_i - x)\,\sqrt{2}\cos(45 - \beta_i)/x$$

$$\epsilon_{s1} = 1.09 \times 10^{-3}$$

Determine the modified value for strain at the concrete surface,
where

$$\epsilon' = K_1\epsilon_1 - K_2 bhf_t/E_sA_s\epsilon_s\gamma_m$$

and $K_1 = 2.3$ for inclined reinforcement
$K_2 = 0.3 \times 10^{-3}$
$\gamma_m = 1.0$

$$\therefore \quad \epsilon' = 2.12 \times 10^{-3}$$

e) The crack width is determined from the lesser of

$$w_1 = \sqrt{2}(a - 0.5y)\epsilon' \quad \text{or} \quad w_2 = 3a_{cr}\epsilon'$$

For this example a_{cr} is based on the maximum spacing between the bars of 140mm, a_{cr} = 73.2mm. [$a_{cr} = \sqrt{(70^2+(35+10)^2)} - 10 = 73.2$mm]

$$\therefore \quad w_1 = 0.76\text{mm or } w_2 = 0.47\text{mm}$$

Therefore the crack width is taken as 0.47mm. Permissible crack width from Table 1 of BS 5400; Part 4 is 0.25mm.

As the crack width exceeds the permissible value, inspection of the half-joint should be undertaken to confirm the condition of the joint.

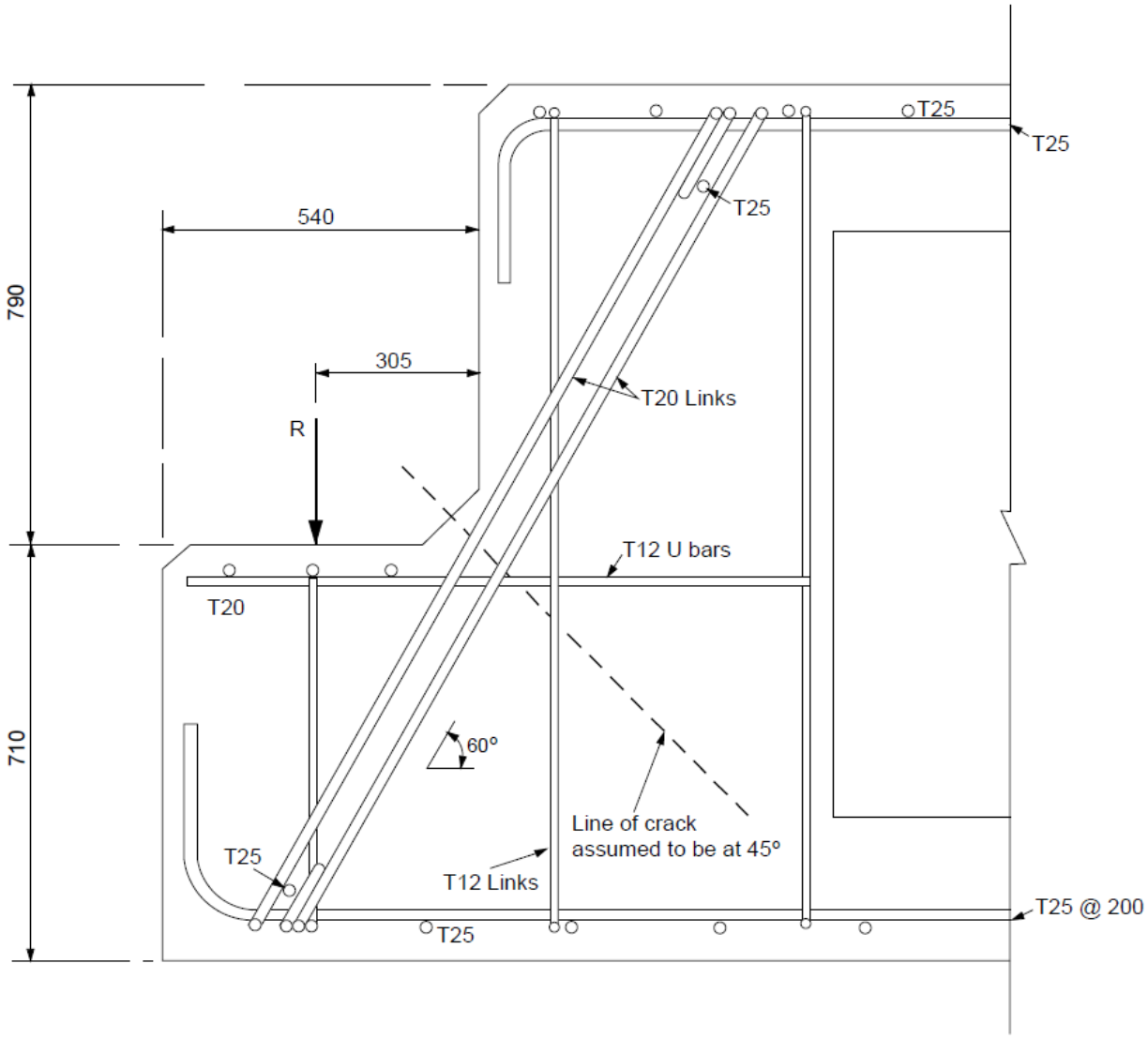


Figure B1 – Half-joint in RC voided slab

APPENDIX C - MANAGEMENT STRATEGY FOR REINFORCED CONCRETE AND STEEL/CONCRETE HALF-JOINTS

Introduction

This appendix gives advice on a management strategy for reinforced concrete and steel concrete composite half-joint deck detailing in suspended span and propped-cantilever bridges. It does not include steel to steel half-joint decks details. It is necessary to ensure that all structures of this type, which are particularly vulnerable to deterioration and difficult to inspect, are recorded, specially inspected, and remedial works planned, to allow the future maintenance funding requirements to be identified.

Background

Half-joints were introduced into bridge decks as a means of simplifying design and construction operations. This form of joint is vulnerable to deterioration in the event of deck expansion joint failure, where chloride rich seepage through the joint can cause concrete deterioration and corrosion of the reinforcement. Loss of reinforcement section through corrosion, or associated concrete spalling can induce higher stresses and significantly reduce the safety margins expected of serviceable structures. Half-joints are a particular concern because they are not easily accessible for inspection or maintenance and they are mostly located over or under live traffic lanes.

Initial Special Inspection

Initial special inspections should determine whether there is evidence of failure of the expansion joint over the half-joint and consequent leakage of water and chlorides on to the bearing shelf of the half-joint. It should also determine whether there is cracking at the re-entrant corners of the half-joint (shown at Annex B), and if present and possible, the width of the crack. The measurements should be averaged to ensure that a true value for the crack width is reported. Care should be taken in the measurement of cracks to avoid overestimation by recording surface effects such as fretting of the concrete surface adjacent to the crack. Bridge temperature should also be recorded since rack width may be influenced by seasonal temperature variation. The severity and location of any other defects such as leaching, or corrosion products should also be recorded, and any relevant concrete delamination and spalling in the vicinity of the half-joint. Whilst carrying out the special inspection, consideration should be given to install monitoring demec pips across the re-entrant corner cracking where there is evidence of significant cracking, to enable periodic monitoring of future changes to crack width.

Further Special Inspection

Where the half-joints have significantly cracked (defined as crack widths >2mm), or where there is evidence of current or past significant seepage, or serious delamination of concrete in the vicinity of the joint, the opportunity should be taken to determine the condition of the reinforcement (if practical). One method of doing this without significant intrusion is to carefully drill small holes to the reinforcement and inspect bars using a borescope, however this will only produce limited information. At the same time if there is significant seepage, limited concrete condition testing (chloride content, cement content, half-cell measurement etc.) should be carried out at the half-joint, if required to supplement existing data already available from earlier principal or special inspections.

Where there is no indication of significant cracking of the half-joints, seepage, or other defects observed, no immediate action is required. However, normal inspection and maintenance arrangements will apply.

Where significant cracks have been observed, and there may be other deterioration, a regime of periodic monitoring and inspection should be instigated. This should generally be based on a visual approach that will target the key factors affecting half-joint performance, such as changing condition, material deterioration or

bridge movements. In some cases it may be appropriate to utilise technical monitoring using strain or other movement gauges. The intervals for monitoring should be appropriate for the structure (eg. 3 months to 1 year), depending on the nature and severity of the deterioration, and the potential risk to the network. The objective of the monitoring is to determine if there is any:

a) progressive horizontal and vertical movement at the joint,

b) movement due to traffic loading and,

c) ongoing material deterioration.

Depending on the ease of access, monitoring of cracks at the re-entrant corner of half-joints can be undertaken on site using a demountable strain gauge to measure manually between demec pips bonded either side of the crack. Manual monitoring is perhaps best used as part of an initial investigation into structural performance. To enable prior warning of structural problems, automatic or remote monitoring using vibrating wire strain gauges is also possible. Embedded silver/silver chloride/potassium chloride half-cells may be used to monitor for potential corrosion risk of reinforced concrete elements.

Proposals for monitoring should be discussed and agreed with the NRA.

Invasive Inspection and Non-destructive Testing

Detailed structural assessment requires accurate information on the condition and geometry of half-joints – this can only be obtained by detailed measurements, invasive inspection, testing and non-destructive methods. Full advantage should be taken of NDT techniques. If it is considered that there is still insufficient information about the condition of the half-joint and its reinforcement for assessment purposes, further invasive testing to expose the reinforcement may be necessary. Such investigations will be subject to technical approval procedures and must be supported by a full technical appraisal, to safeguard the structure during the course of the work, and to set down the type of investigation proposed, and details of the expected outputs.

Consideration should be given to selecting the most appropriate bridges for invasive testing, and the most suitable test location(s) on the bridge. Where invasive testing involves de-stressing the half-joint reinforcement, the additional loading carried by the adjacent bar sets should be assessed, and the necessity and effects of propping the bridge during the work considered. In determining testing locations, concentration of half-joint loading, drainage paths and the severity of defects should be considered, together with safety, access and traffic management issues.

Detailed proposals for invasive testing should be submitted to the NRA 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 reinstatement of test areas, and the selection of materials, method of reinstatement, given the time constraints, weather 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 half-joint reinforcement, unexpected delays and weather conditions.

Non-destructive testing methods such as impact echo, radiography, acoustic emission, and thermography etc. may be considered to minimise the need for invasive inspection of half-joints. Whilst NDT methods alone are unlikely to give definitive indications of defects and overall condition, they can be used to assist determination of the variations in condition along joints, and may 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 the relative condition elsewhere, or point to where further invasive inspection may be necessary. However some care is required in selection of the NDT technique, as the difficulty in access, health and safety issues, and unsuitability of application to half-joints may prevent their widespread adoption, and the production of meaningful data. However most of the NDT techniques are either still under development in terms of robust and reliable site equipment, and/or have not been used on half-joint decks, so there will be little in the way of comparative site data available.

Structural Assessment

Where half-joint structures are exhibiting significant deterioration, existing structural assessment reports shall be reviewed as part of the Assessment, and new assessments carried out as appropriate.

Particular attention should be paid to the method of analysis previously adopted, and whether it is still considered appropriate: any assumptions made about the condition of the half-joint in the assessment, and the continued appropriateness of any departures from standards previously granted. It is recognised that previous assessments concentrated on the effects of the 40 tonne 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, and particularly the condition of the half-joint.

Assessment should be carried out in two parts:

a) To determine the range of load effects on the half-joint;

b) To calculate the capacity of the joint in its deteriorated condition, the Structural Assessment Engineer should use their judgement as to the deteriorated condition of the joint taking account of the likely loss of reinforcement section and the effects of delamination of cover concrete. Reference should be made to NRA BA39 'Assessment of reinforced concrete half-joints' as necessary to assist

One of the objectives of the assessment should be to identify a deterioration trigger point to feed into a monitoring and inspection regime, and to assist in determining when interim safeguarding 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 reinforcement yielding, 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': the ratio of load effect to assessed joint capacity. Technical Approval procedures in accordance with NRA BD 2 will apply to this assessment work.

Maintenance

For all half-joint bridges, high priority should be given to preventing further deterioration of half-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 prioritised as essential maintenance, and submitted as part of the normal funding arrangements. Advantage should also be taken during any planned re-waterproofing or resurfacing work to undertake inspection and concrete condition testing of half-joints, and reinforcement inspection from above.

Expansion joint replacement and renewal of waterproofing (where they have shown to have failed) are the most important preventative remedial actions to safeguard against further deterioration of a half-joint.

Repair

The repair of half-joints is made particularly difficult due to poor access, generally congested reinforcement and traffic management issues. Advice is given below on possible repair methods.

Concrete replacement is an option for repairing deteriorated concrete. Information on concrete replacement is provided in NRA BD 27 'The Repair of Concrete Road Structures'. Unless such practices are adopted, it is likely that concrete repairs will be only partially effective in minimising future corrosion of reinforced concrete.

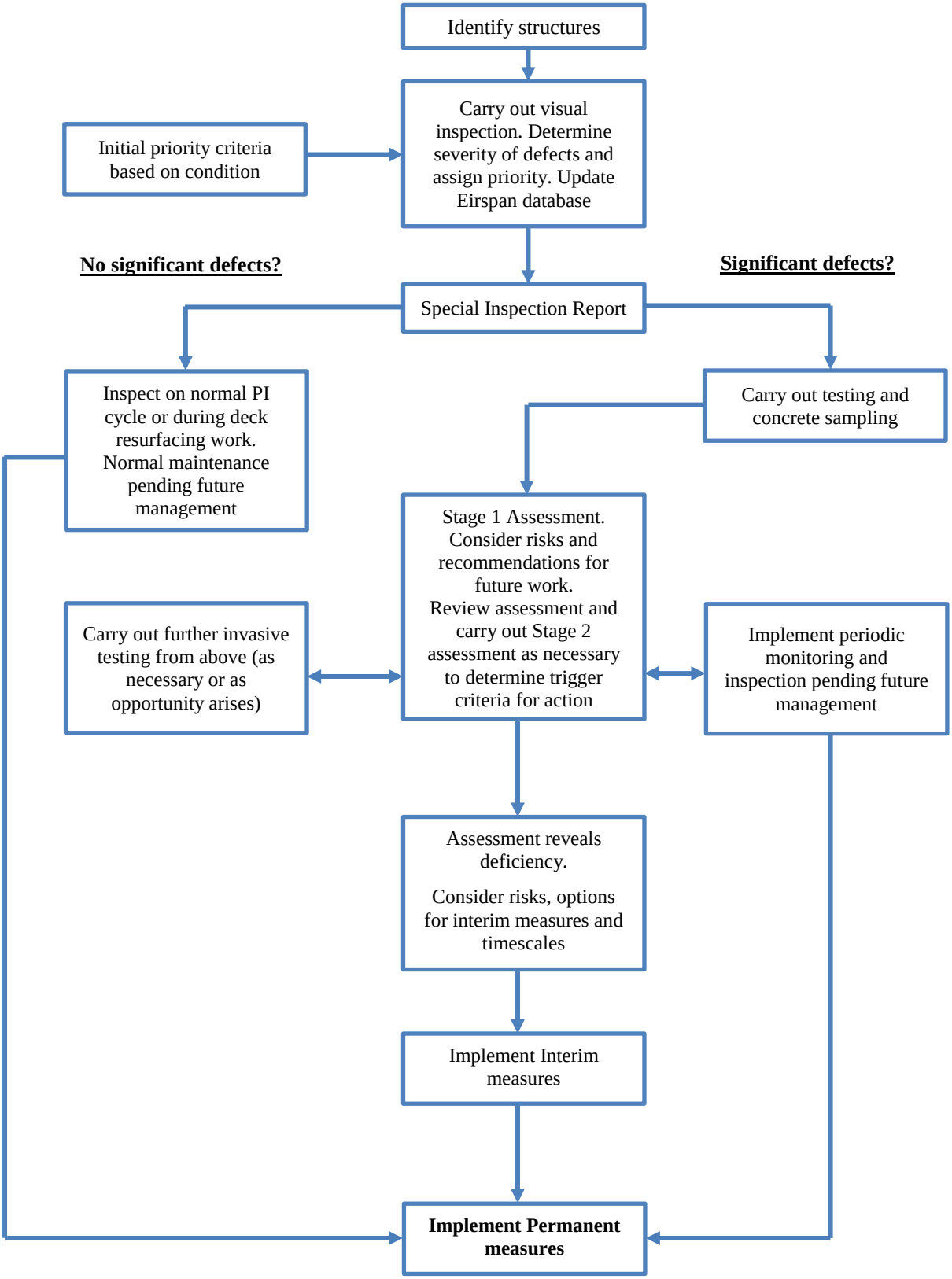
Information on cathodic protection (CP) is available as an Advice Note NRA BA 83 'Cathodic Protection for Use in Reinforced Concrete Road Structures'. This can be an effective technique for minimising future corrosion in reinforced concrete, usually in combination with some concrete repair work. However it is essential that specialist advice is sought if cathodic protection is to be considered. It is also important that the

condition of the half-joint and in particular the reinforcement is known with certainty. CP is an active corrosion control method, but it must be managed and monitored to ensure continued effective operation. If it is, then there should be no further deterioration to effect the load capacity of the half-joint.

Where half-joints have deteriorated so badly that it is practically or economically beyond repair, such as the reinforcement is so badly corroded that it cannot be satisfactorily reinstated, then replacement of a whole element may be a cost effective option.

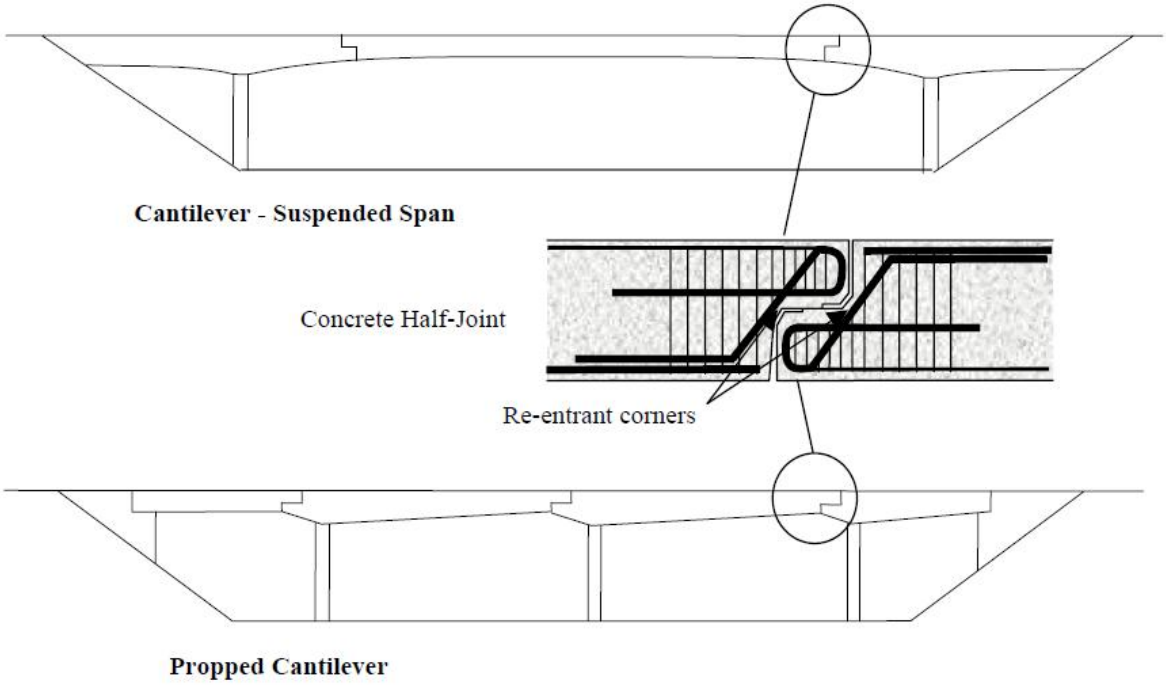
There are a number of alternative commercial repair systems available to manage deteriorating reinforced concrete such as, chloride extraction, galvanic protection, and active moisture reduction systems. The effectiveness of these particular remedial methods for use on half-joints is not yet proven and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

ANNEX A – MANAGEMENT STRATEGY

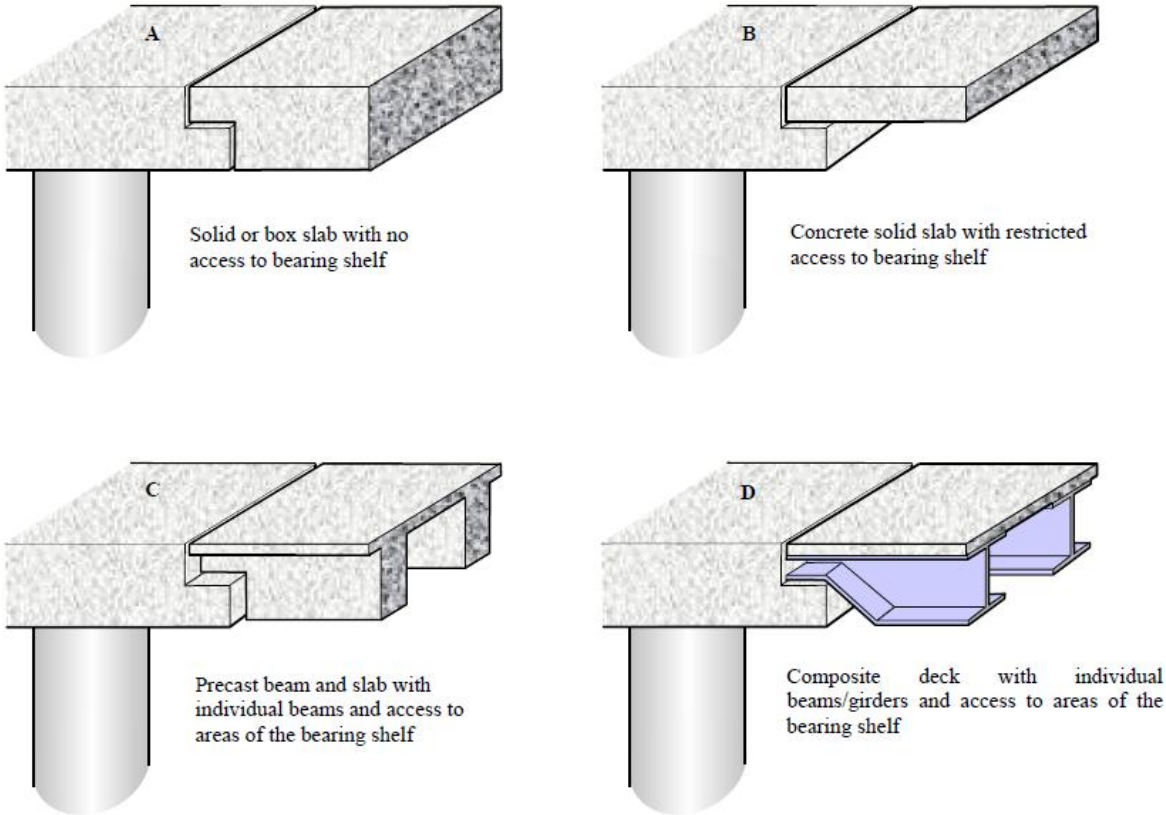


Management Strategy for Concrete Half-Joint Deck Structures

ANNEX B – TYPICAL HALF-JOINT DETAIL



Typical half-joint details



Types of half-joint categorised by access to bearing shelf

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