

HIGHWAYS AGENCY
CONTRACT 2/419 TECHNICAL AUDIT OF THE APPLICATION OF BA79

A REVIEW OF BRIDGE ASSESSMENT FAILURES ON THE MOTORWAY AND TRUNK ROAD NETWORK

Final Project Report

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EXECUTIVE SUMMARY

A sample of 294 structures has been audited in order to appraise the results of the assessment element of the Assessment and Strengthening programme.

Auditors have collected the data from 7 different Highways Agency managing agents and 7 further offices in Scotland, Wales and Northern Ireland. 20,995 pieces of data were recorded in a purpose-designed database.

Analysis of the data has been carried out, resulting in the identification of the following:

- the relative incidence of each type of structure failing assessment
- critical elements, locations and potential modes of failure
- the reasons for assessment failures.

The results of the analyses have been presented graphically and numerically. The relative incidence of assessment failures has been presented, with reference to deck material types, deck construction types, and construction dates. Critical elements, locations and modes of failure have been identified, with separation of non-carriageway failures due to accidental verge loading or impact loading of parapets and piers.

The reasons for assessment failures have been categorised into the following factors:

- Increases in loading, so that the assessment loading is greater than the structure was originally designed for.
- Updates in the standards in response to research, so that the assessment capacity is less than the original design capacity according to the design code of the time.
- Inappropriate or too conservative analysis for assessment.
- Lack of design information leading to conservative assumptions.
- Misinterpretation or inappropriate application of the assessment code.
- Reduced capacity due to deterioration or damage.
- Poor original design or construction.

Each of these factors has been analysed, including a review of the changes to the standards for loading, design and assessment. Particular reasons for assessment failures have been investigated and highlighted in the results.

On the basis of these results and other research, recommendations have been made for possible revisions to technical standards. Particular recommendations include revision of the method of assessment of the shear capacity of concrete bridges with poorly anchored reinforcement. In addition, it is recommended that approaches to enhance the level of technical understanding brought to bridge assessment be embraced.



ACKNOWLEDGEMENTS

This report was prepared by Parsons Brinckerhoff Ltd under contract to the Highways Agency.

Both Parsons Brinckerhoff Ltd and the Highways Agency would like to thank all those organisations and individuals who gave so freely of their time by:

- responding to the questionnaire,
- providing data on their sub-standard structures,
- agreeing to be interviewed and
- allowing the audit teams access to their structural records.

It is recognised that without their cooperation, this project could not have been successfully completed.

1 INTRODUCTION

In November 1987 the then Minister for Roads and Traffic announced a 15-year programme of bridge rehabilitation (“the bridge programme”) with the aim of bringing road structures up to current standards. The programme comprised three main sub-programmes, of which assessment and strengthening was one.

Where structures failed their initial assessment there was a requirement to undertake further investigations and further assessment calculations. The complexities of the further work varied from structure to structure.

In considering assessment results, preliminary observations of the results of the Assessment and Strengthening programme gave cause for concern about current design and maintenance policy in the following areas:

- Many bridges with concrete elements were assessed as substandard for reasons other than shear deficiency or deterioration in condition. (It had originally been anticipated that these would be the principal reasons for assessment failures).
- Very few of the assessment failures were adequate for the pre-40 tonne family of vehicles.
- Many structures that passed their assessments had little reserve strength above the current requirements and in many cases, shear or condition deficiencies were not the critical considerations.

In August 2002 Parsons Brinckerhoff (PB) was commissioned to undertake an audit of the application of Highways Agency Advice Note BA79, “The Management of Sub-Standard Highway Structures”, together with an appraisal of the results of the assessment element of the Assessment and Strengthening Programme, under Contract 2/419.

The sample of structures used in the appraisal of the results of the assessment element of the Assessment and Strengthening Programme was drawn from:

- The Highways Agency,
- The Scottish Executive Development Department,
- The Welsh Assembly Government,
- The Department for Regional Development for Northern Ireland.

This report sets out the findings of the audit and appraisal of the assessment element of the Assessment and Strengthening Programme. The methodology of the data collection is detailed in Section 3, while Sections 4,5 and 6 describe the results that were obtained. Section 7 considers possible revisions to current policy and practice. Conclusions and recommendations are included in Section 8 and Appendix A contains further guidance on the realistic assessment of concrete structures.

2 OBJECTIVE

The objective of the appraisal of the results of the assessment element of the Assessment and Strengthening Programme was to capture the reasons for failure, with the aim of using the findings to inform future assessment policy and practice and possible revisions to technical standards.

3 METHODOLOGY

3.1 Preliminary data collection

The appraisal of assessment results was carried out using a phased approach, with the initial phase seeking to establish a matrix of the different forms of structures on the trunk road network in England, Scotland, Wales and Northern Ireland assessed as substandard.

In addition, during the first phase a review was undertaken to examine the key changes in design and assessment standards. For each of the different structure types assessed as substandard, a matrix of the potential forms of assessment failures was developed together with analytical approaches that might be used in such cases to achieve realistic assessment ratings with criteria relevant to their application.

For each of the different structure types and potential forms of assessment failures, a matrix was developed summarising the sensitivity of these forms of potential failure to structure condition together with relevant indicators that could be identified during an inspection.

Bridge owners were also contacted to request the information required to develop a matrix of sub-standard structures to be used to inform the audit selection process.

For their sub-standard structures, the bridge owners were requested to provide the information presented in Table 3.1.

- Structure No
- National Grid Reference
- Year commissioned
- Is the structure on a heavy load route?
- Design load
- Design standard (and version)
- Any special design loading
- Type of deck construction
- Form of deck
- Nature of end supports
- Nature of intermediate supports
- Nature of foundations
- Assessment rating
- Assessment standard (and version)
- Level of assessment (as defined in BA79)
- Reason for deficiency
- Is the structure managed to BA79?
- Are interim measures in place?

Table 3.1 – Structural Information requested

In general, due to the nature of each organisation's bridge management system they were unable to provide the entire list of data requested. More commonly, PB was provided with the bridge name and number, date of construction (where known), the type of deck construction and the assessment rating¹.

¹ Any missing information was collected during phase 2.

The distribution of requests for the information and the responses received is shown in Table 3.2.

Organisation	No. requests for info	
	Submitted	Returned
7 No HA MAs	7	7
Devolved Authorities	3	3
Local Authorities	15	6
Total	25	16

Table 3.2 – Information on sub-standard structures

3.2 Sampling

Upon receipt of the data from bridge owners it was collated within an electronic database. The database was purpose-developed to facilitate the selection of a representative sample of substandard structures for audit. The selection process was designed to ensure all major forms of construction and structure types were represented over a wide range of bridge owners.

Each of the structures selected for audit was assigned an audit category to signify their presence on the *primary* audit list. The remaining structures were assigned to a *secondary* audit list to indicate that that they would only be audited if one of the primary selection proved to be unsuitable, unavailable for audit, or if there was additional time available during the audit to increase the sample number.

3.3 Technical audit

In order to facilitate consistent data collection and maximise efficiency, a further database was developed to record the results of each structure audit. The database incorporated the following key features:

- User friendly interface to minimise the learning curve associated with its use
- Standards menus, pull-downs etc. to make data entry efficient and reporting consistent
- Assigned data fields to enable all data described in the project brief to be collected and analysed, together with additional data fields to facilitate the capture of any relevant supplementary information to meet the wider aspirations of the brief.
- Pre-populated with data on sub-standard structures supplied by participating bridge owners.

3.3.1 Auditor training

Prior to the audits being undertaken, all auditors took part in a detailed training session. The training covered the following aspects:

- Bridge Inspection and Assessment
- Analysis Methods
- Review of Loading Standards
- Review of Design and Assessment Standards
- Bridge Deterioration
- Common reasons for assessment failures
- Methods of achieving more accurate assessments
- Database Operation
- Audit Procedures

3.3.2 Audit visits

Each audit team comprised between 2 and 4 auditors, the number being tailored to suit the requirements of the bridge owner, the number of structures in the sample and the time available to carry out the audit.

Each auditor was equipped with a laptop computer containing an individually numbered copy of the database. By numbering each copy it was possible to carry out internal quality reviews of the data being collected and clarify any areas where the results were unclear.

The data collected on the audit visits was extracted from the information that was made available. The information that was requested is shown in Figure 3.1, with each pie chart giving the proportion of audits where the information was provided. The quality and quantity of the records varied from visit to visit, and the auditors extracted as much relevant data as possible. Sometimes this involved some interpretation and judgement on the part of the auditors, particularly when determining the reasons for assessment failures.

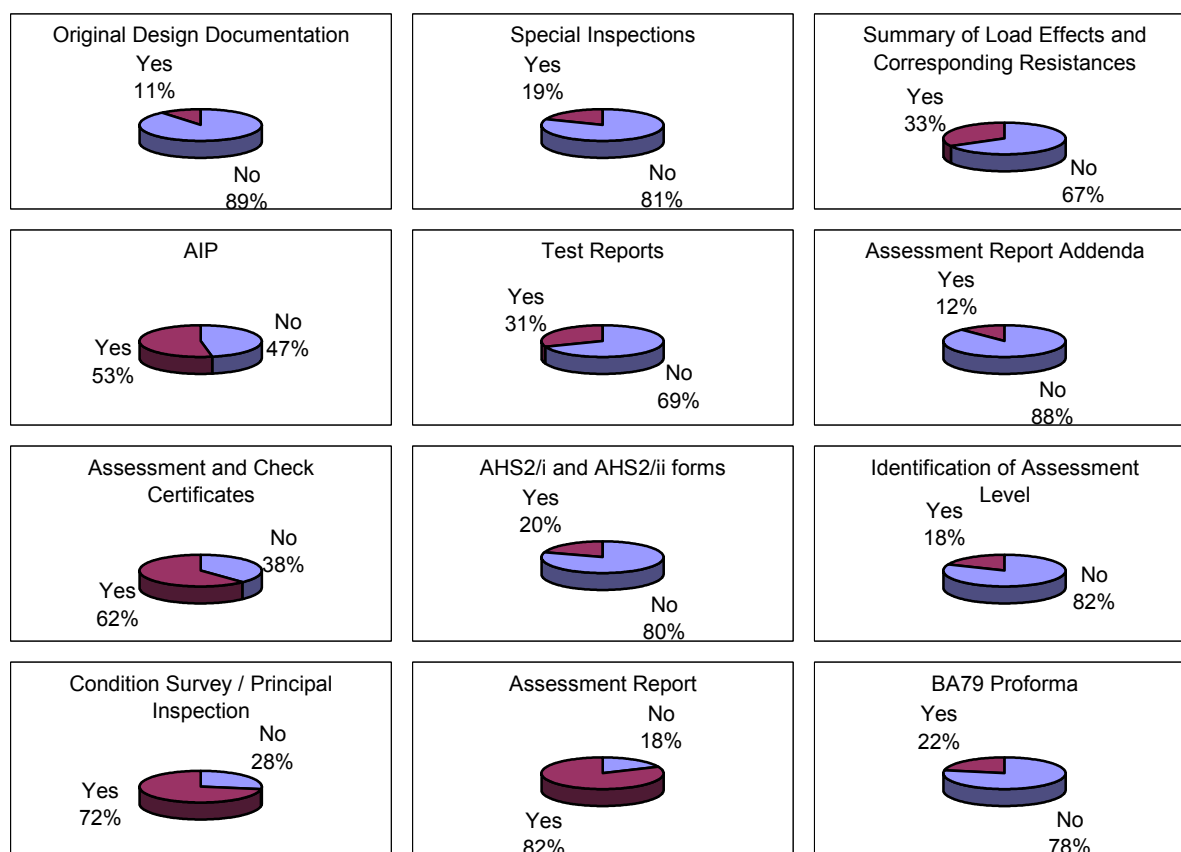


Figure 3.1 - Documentation provided for audits

The total number of sites visited was 14, and the total number of structures audited was 294. The number of structures from each organisation is shown in Figure 3.2, compared with the number of structures on the provisional primary list of structures to be audited that was agreed in the Interim Report.

In practice, it was only possible to audit the structures that were made available, and on some occasions the number of structures provided fell short of the number requested. In addition, it

emerged that some of the structures had passed assessment and were not substandard, and so did not fall within the scope of the audit. However, it was agreed that structures on the list that had been subsequently strengthened would be audited for their last assessment before strengthening.

A secondary list of structures had also been requested to compensate for the possibility of structures on the primary list being unavailable for audit. On some occasions it was possible to audit a greater number of structures than planned, by auditing structures on the secondary list. On other audit visits, even with the secondary list it was not always feasible to audit the planned number of structures. Due to the inconvenience of providing the records for audit, the organisations were generally reluctant to provide any more structures than necessary. One particular organisation only provided those structures they deemed to be relevant to the brief, which turned out to be fewer structures than requested on the primary list.

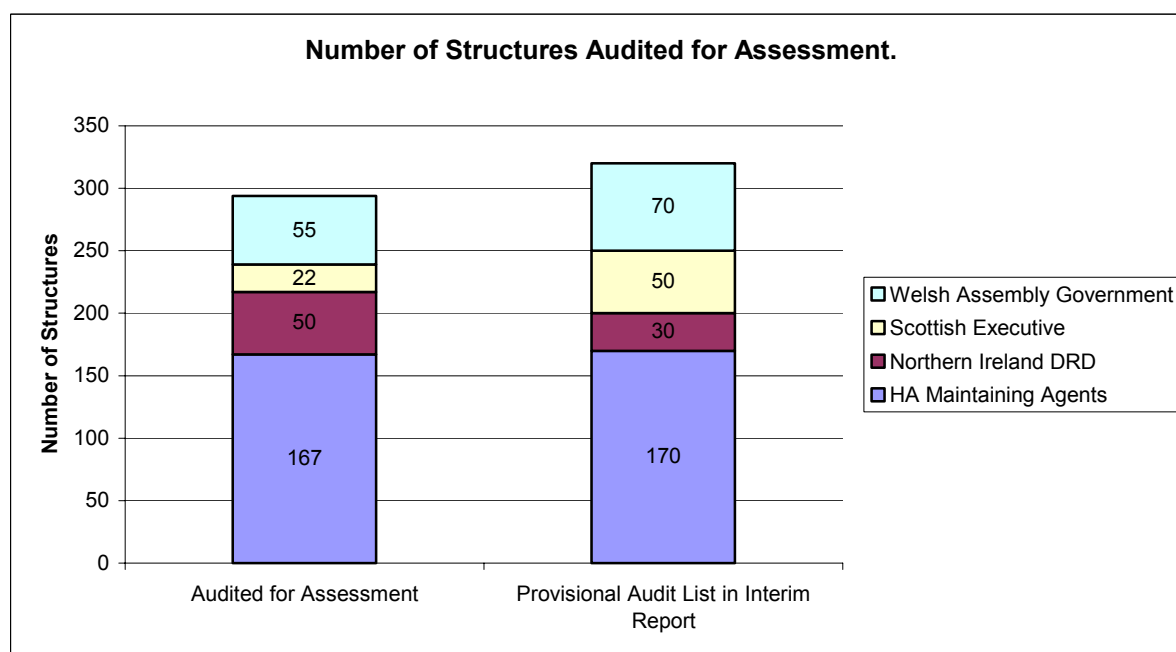


Figure 3.2

4 RELATIVE INCIDENCE OF ASSESSMENT FAILURES

The structures that were audited have the properties shown in Figures 4.1 to 4.3. The structure types for the audit sample were selected to reflect the stock of structures assessed as substandard in each country's network. The deck material types are illustrated in Figure 4.1, and the deck construction types are illustrated in Figure 4.2. The most common form of structure failing assessments is a reinforced concrete slab.

The dates of construction of the structures audited are illustrated in Figure 4.3. The majority of the structures audited were constructed between 1960 and 1975, but structures of other ages were also reviewed in order to obtain as much data as possible on the full spectrum of structures failing assessments.

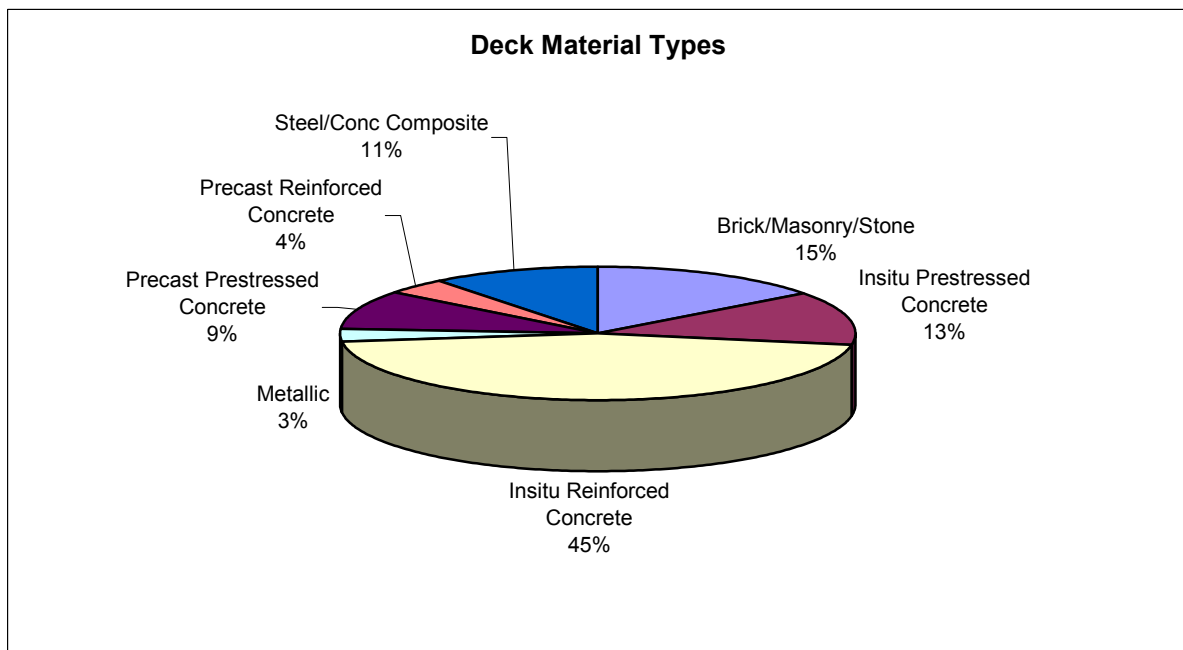


Figure 4.1

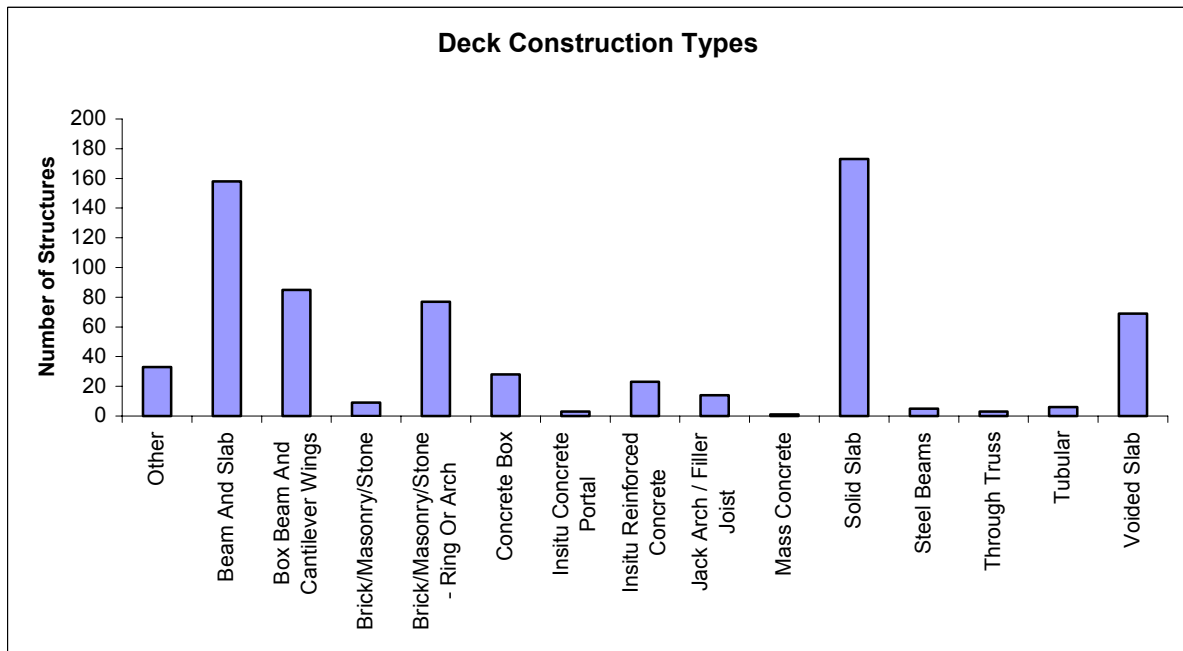


Figure 4.2

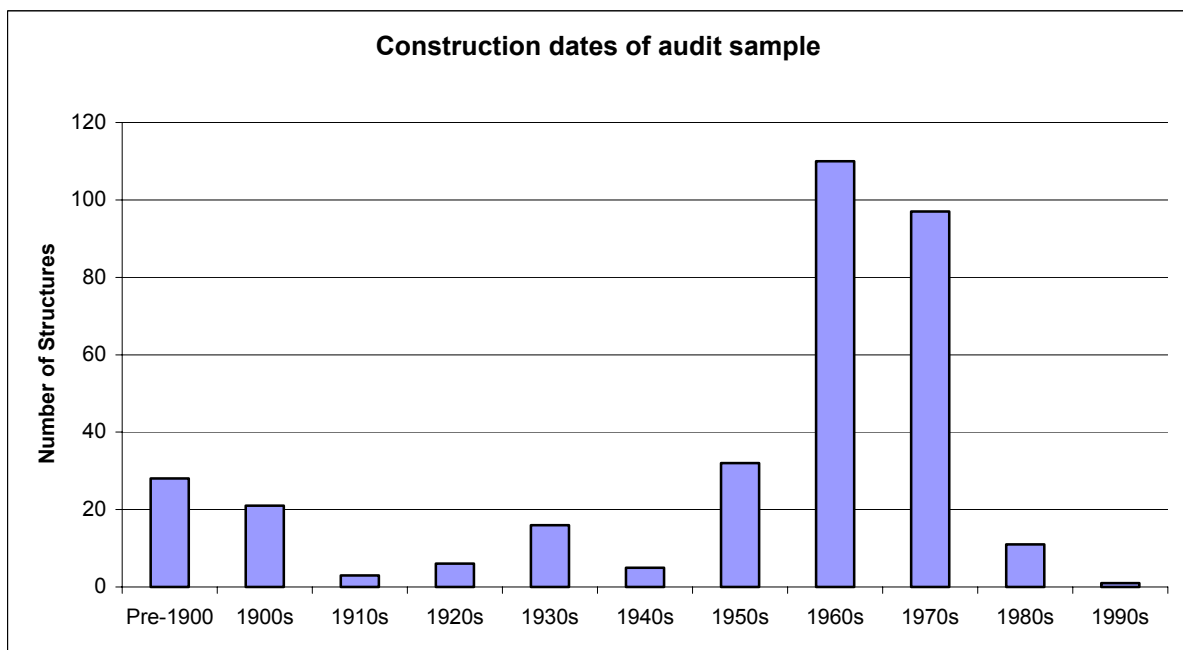


Figure 4.3

5 CRITICAL ELEMENTS, LOCATIONS AND POTENTIAL MODES OF FAILURE

5.1 Critical elements

The substandard structures in the audit were categorised by the elements that had proved to be critical in their last assessment. Where more than one element appeared to be critical, all critical elements were recorded. These results are illustrated in Figure 5.1. The most common substandard elements are slabs, main beams and piers, followed by deck cantilevers and parapets.

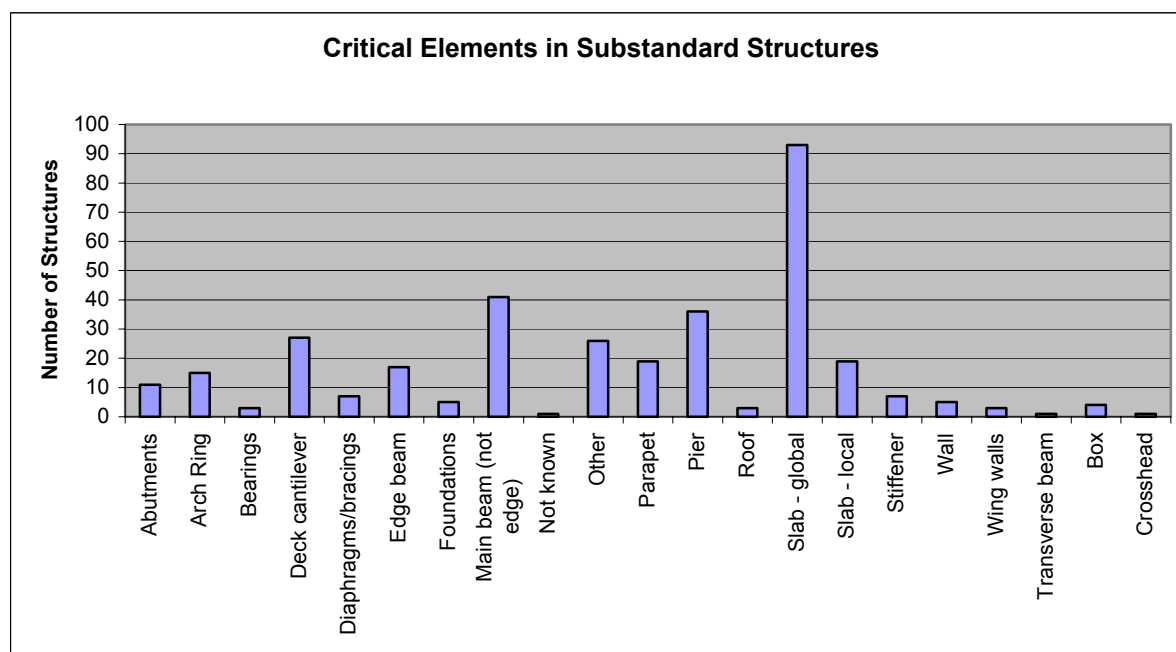


Figure 5.1

5.2 Critical Failure Modes

The predicted failure modes associated with the critical elements from Figure 5.1 are given in Figure 5.2. The most common modes of failure are longitudinal flexure, transverse flexure and general shear. The auditors chose the “not known” field where the provided information did not specify the mode of failure.

For concrete slabs, the critical modes are also shown separately in Figure 5.3, and the same three major modes of failure dominate. While shear in concrete might be expected to be a problem due to the changes in design rules, the methods for flexural design have not changed in the same way. The large amount of flexural assessment failures is more likely to be associated with the method of analysis or the increase in loadings. These factors are considered in more detail in Section 6.

5.3 Assessment ratings

The assessment ratings for the audited structures are illustrated in Figures 5.4 and 5.5. The ratings shown refer to the carriageway only. Structures in the audit sample that had a 40 tonne carriageway rating generally had substandard piers or parapets, or weak cantilevers or verges that could not take accidental vehicle loading. These secondary failures are illustrated in Figure 5.6. The secondary elements were less likely to have been recorded as substandard for structures with carriageways with lower assessment ratings, as illustrated in Figure 5.6(b). This may be because elements that could fail due to impact or accidental loading were judged to be of lower priority than failures of the carriageway.

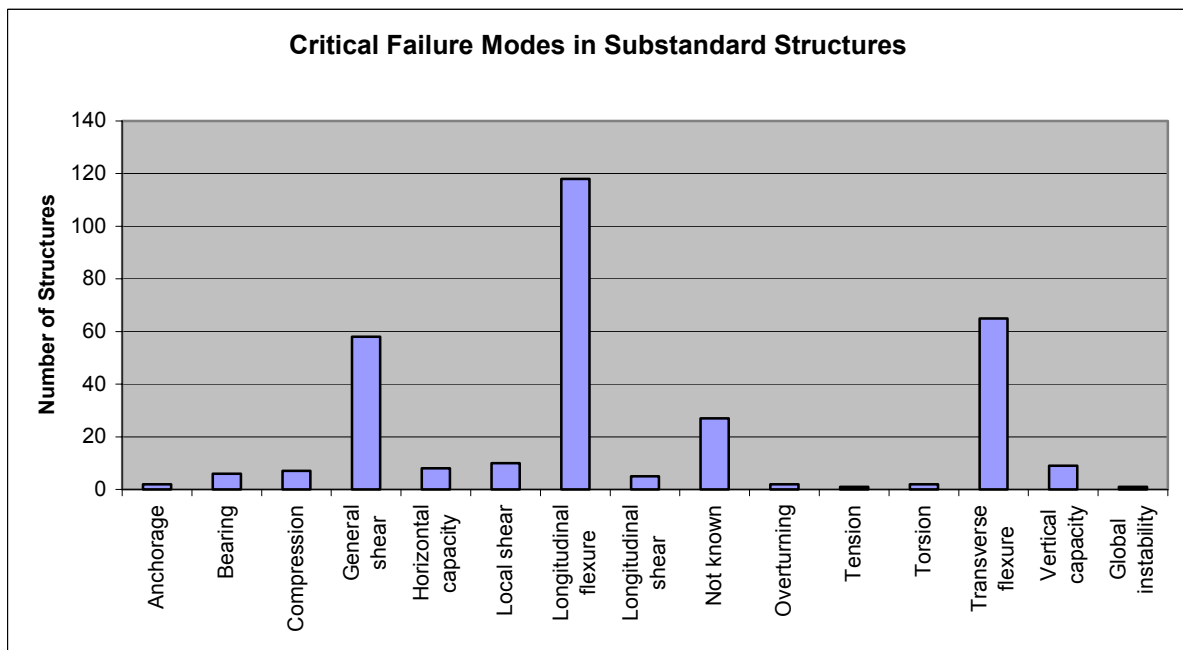


Figure 5.2

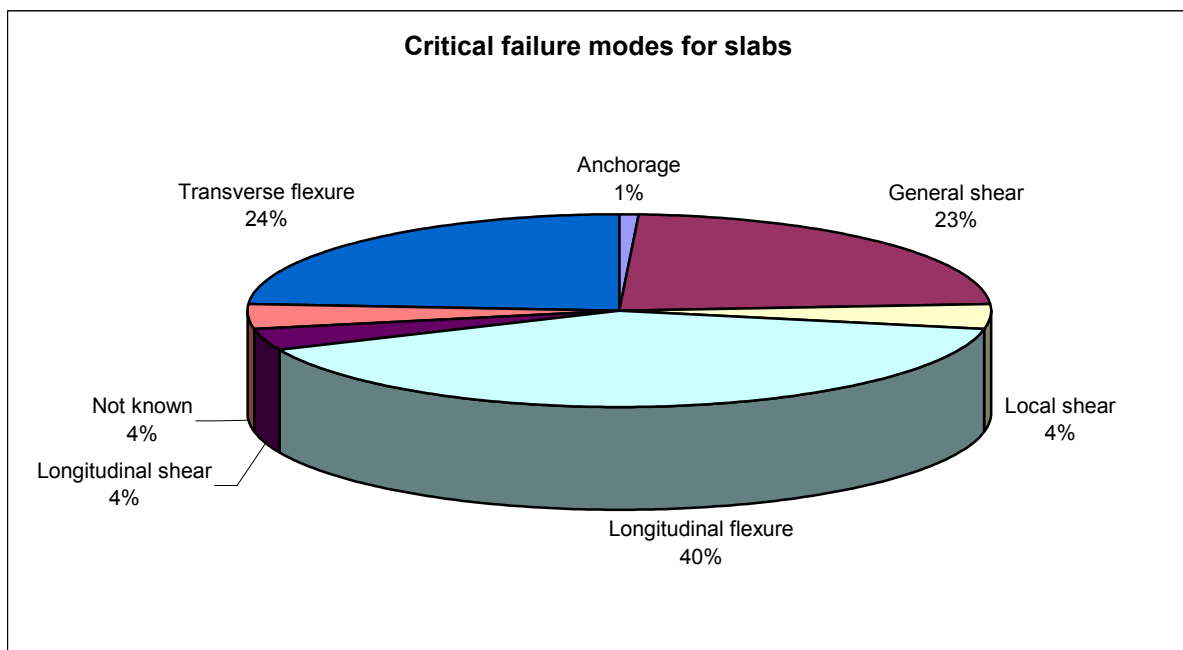


Figure 5.3

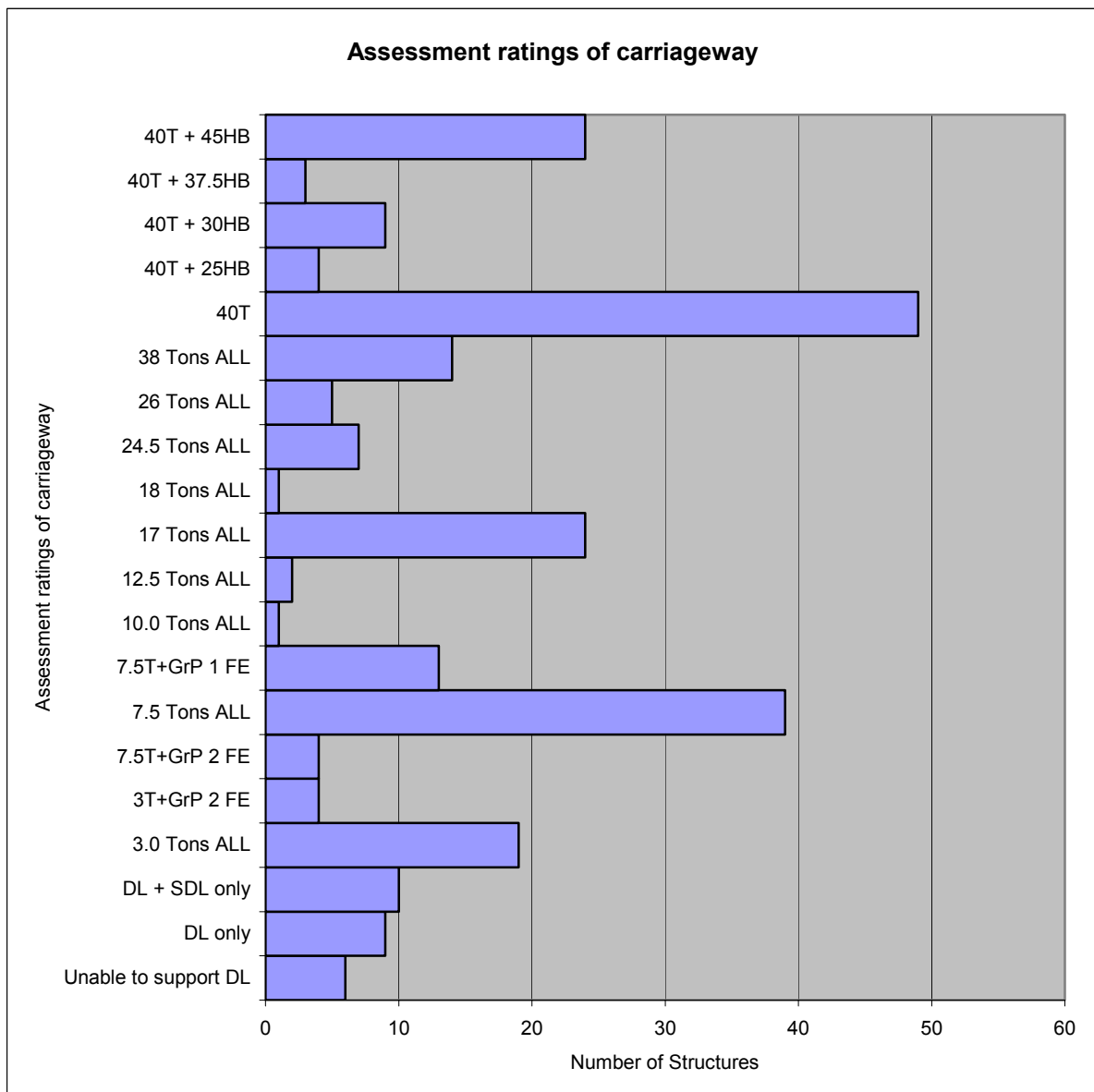


Figure 5.4

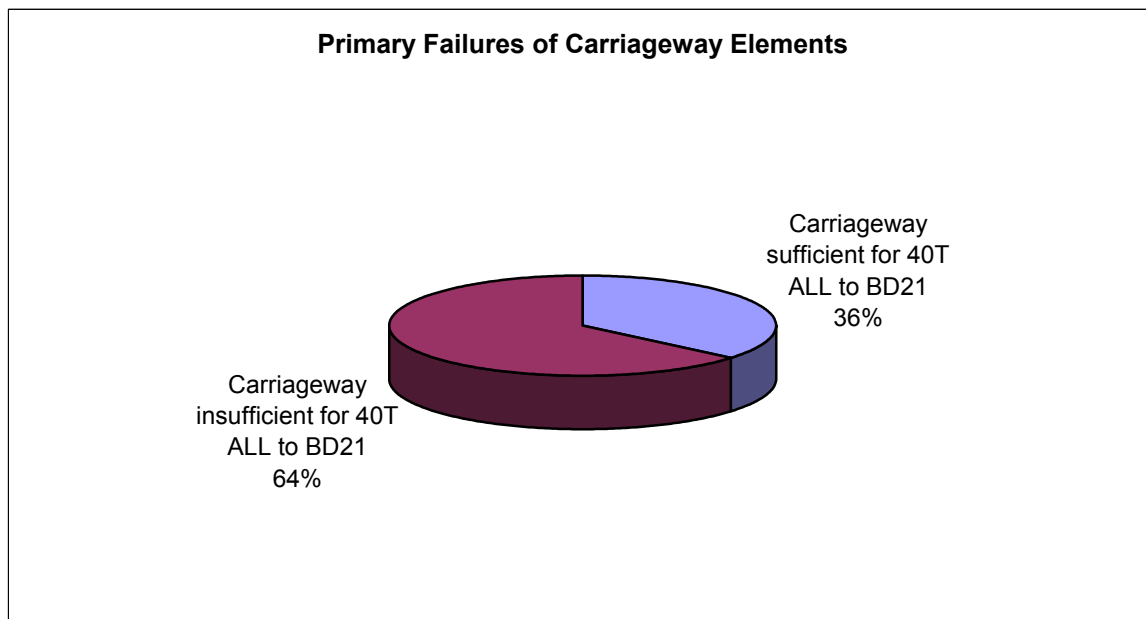


Figure 5.5

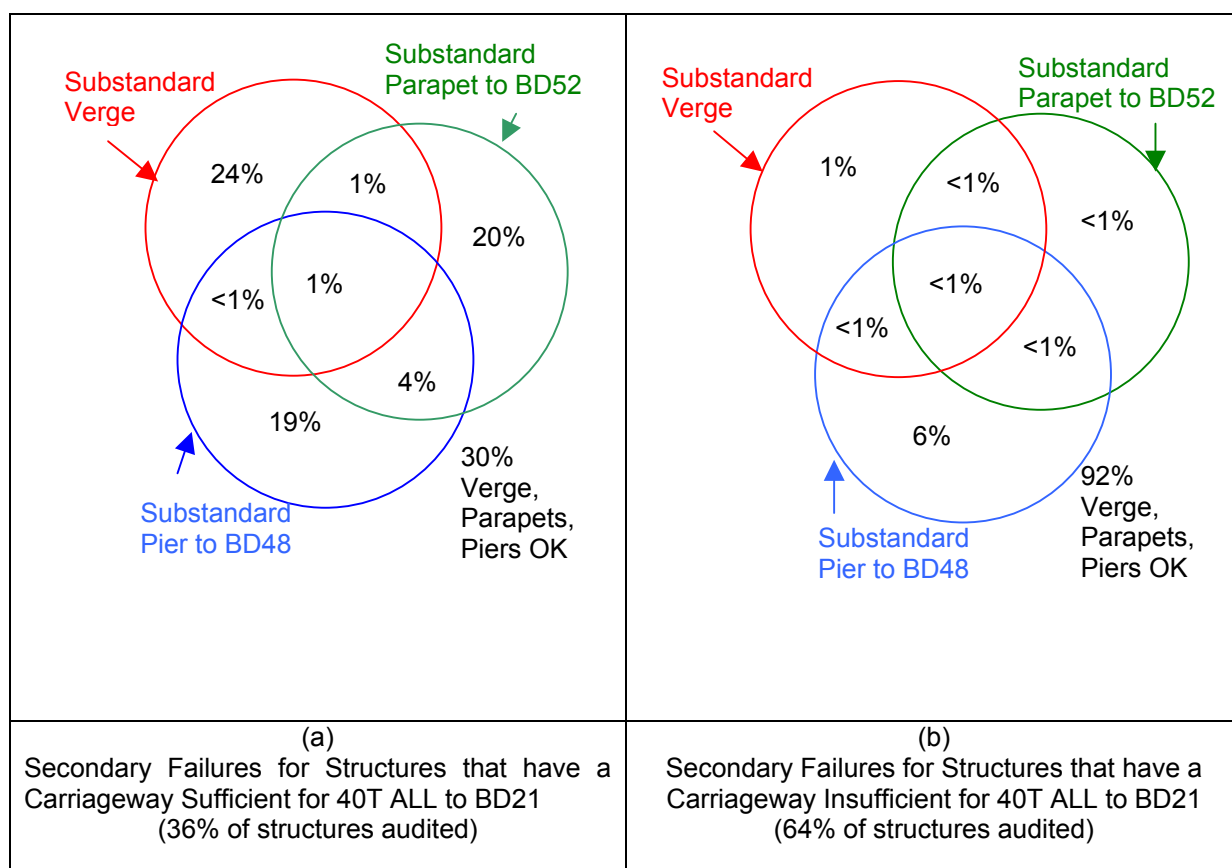


Figure 5.6

6 REASONS FOR ASSESSMENT FAILURES

6.1 Introduction

Assessment failures can usually be considered to be caused by one or more of the following factors:

- Increases in loading, so that the assessment loading is greater than the structure was originally designed for.
- Updates in the standards in response to research, so that the assessment capacity is less than the original design capacity according to the design code of the time.
- Inappropriate or too conservative analysis for assessment.
- Lack of design information leading to conservative assumptions.
- Misinterpretation or inappropriate application of the assessment code.
- Reduced capacity due to deterioration or damage.
- Poor original design or construction.

In practice, assessment reports may not always include sufficient information to determine conclusively which of these factors is critical. There may be a combination of reasons why a bridge has failed an assessment, which may not always be immediately obvious without performing a full check on the assessment. However, the auditors, as experienced bridge engineers, were familiar with the issues arising from bridge assessments and were able to make informed judgements based on the information available. If an assessment failure appeared to be caused by a combination of factors that may have been independently critical, then all of the relevant factors were recorded. The training provided to auditors ensured that a consistent approach was taken.

Figure 6.1 shows the number of substandard structures from the audit assigned to each of the reasons for assessment failures. Secondary elements such as piers and parapets that are not assessed to BD21 have been separated in Figure 6.1. Neglecting these secondary elements, the most common reason for assessment failure is conservative or inappropriate analysis; including the secondary elements, the most common reason is increased loading. Since each structure could be placed in more than one of these categories, the numbers in Figure 6.1 do not sum to the total number of structures.

This section explores the background to each of these reasons for assessment failures, with a breakdown of the respective data from the audit.

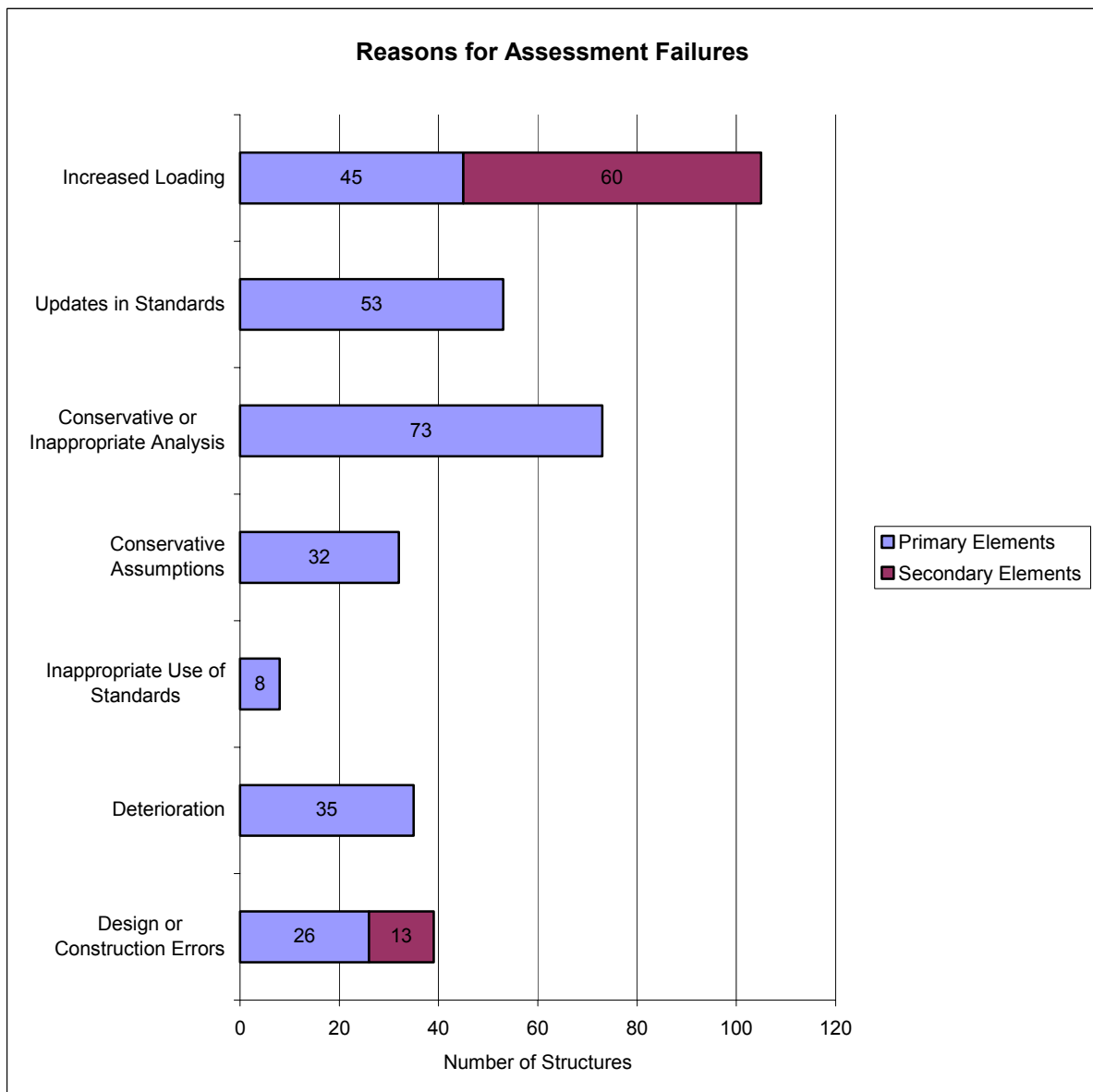


Figure 6.1

6.2 Changes in loading standards

The traffic loads that older bridges were designed to carry may be considerably lower than current loads. Increased loading is a very common reason for older bridges to fail assessments.

6.2.1 Review of loading standards

Loading on highway bridges has consistently increased over the years. The Ministry of Transport introduced the first government load train for highway bridges in 1922, incorporated into a British Standard in 1929. This was updated to an equivalent loading curve in 1931, which was the forerunner to the familiar HA loading, comprising a uniformly distributed load that was a function of the loaded length applied in combination with a point load, allowing quicker analysis.

BS153 Part 3A 1954 included both HA loading and HB loading, which represents an abnormally heavy vehicle. The HA and HB models were retained in the 1972 version of BS153 Part 3A, but revised in the loading standard BE1/77, and revised again in BS5400 Part 2 and BD37/88.

The permitted axle weights and gross vehicle weights have risen consistently, leading to worries about the adequacy of older bridges. The allowable weights had risen to a 10 tonne axle weight limit and a 32 tonne gross vehicle weight limit by 1973, which was increased to 38 tonnes over 5 or 6 axles in 1983. EC directive 85/3 allowed 40 tonnes over 5 axles and an axle weight limit of 11.5 tonnes, which was implemented in the UK in 1999.

The need to assess existing bridges for these increases in loading has been central to the development and subsequent updating of assessment standards.

The first standard to cover highway bridge assessment, BE4, was introduced to allow for 32 tonne vehicles. The increase to 38 tonne vehicles led to the new assessment standard BD21 in 1984, which was amended in 1993 for 40 tonne vehicles. The loads in these standards are based on the design loads, including the worst effects of impact, bunching and vehicle overload, which may be considered to be over-conservative for the purposes of assessment. BD21/97 and the later version BD21/01 therefore relaxed the loading requirements accordingly in clause 5.23.

Annex D of BD21 covers loading of transverse members, buried structures, non-cantilevered footways and verges. In BD21/97 Annex D was somewhat ambiguously written and it was unclear which axles had an associated impact factor. The worst case was generally assumed, with the impact factor of 1.8 applied to both adjacent vehicles, leading to a large concentration of load. This loading was often more onerous than the design loading, so that a newly designed structure could appear to fail an assessment. These inconsistencies have now been addressed and Annex D has been clarified in BD21/01.

The loading for assessment of buried structures has also been clarified and adjusted in Annex D of BD21/01. For load dispersal through the fill, BD21/01 refers to BA55, and BA55/00 refers in turn to BD31/87. The simple dispersal method in BD31/87 can suggest some anomalous, high load concentrations where the dispersal zones overlap. However, BD31/87 also allows the use of Boussinesq's theory of load dispersal, which provides a loading curve based on elastic continuum mechanics and can be significantly less conservative than the simple dispersal method in BD31/87. PB is currently working on an updated version of BA55 that provides guidance on more appropriate assessment methods for buried structures and draws the advice note in line with the updated design standard for buried concrete structures, BD31/01.

There have also been occasions where the assessment standards have introduced more onerous loading requirements, to reflect possible accidental loads that were not previously explicitly covered in design. This can often expose deficiencies in older bridges. For example, many cantilevered footways built before 1960 only appear to have enough capacity for footway loading, and not accidental vehicle loading as required for assessment. Parapets, if required to be assessed, are also often not strong enough for the design loads in BD52 or simply fail geometry requirements, and the parapet stringcourse sometimes has insufficient strength for the parapet loads. The introduction of the standard BD48, which gives pier impact loads for assessment and strengthening, has highlighted that many slender bridge columns may have insufficient strength to withstand an impact from an HGV.

6.2.2 Assessment Failures due to Increased Loading

The loading on most existing bridges will have increased since they were designed, and so many assessment failures might be judged to be at least partly due to the increased loads. However, this factor was only selected where it was judged that the increased loading was a primary reason for failure, even when considering the effect of any other factors. Auditors may have tended to assume this option particularly for structures where there was insufficient information provided to infer any other possible reason. This may, in part, have led to an increased frequency of the increased loading response in comparison to the other options.

Many of the bridges audited had substandard piers, due to insufficient resistance to the impact loads in BD48/93. Parapets were also commonly substandard. However, failures due to increased loading were, in fact, most commonly associated with increased lane loading.

6.3 Changes in Standards for Design and Assessment Capacities

Over half of the assessment failures caused by updates in the standards were associated with the change in the design of reinforced concrete for shear, as illustrated in Figure 6.2. It had been expected that this would be a significant issue, since the design rules changed substantially in 1972. The relevant changes in the Standards are described in the following section.

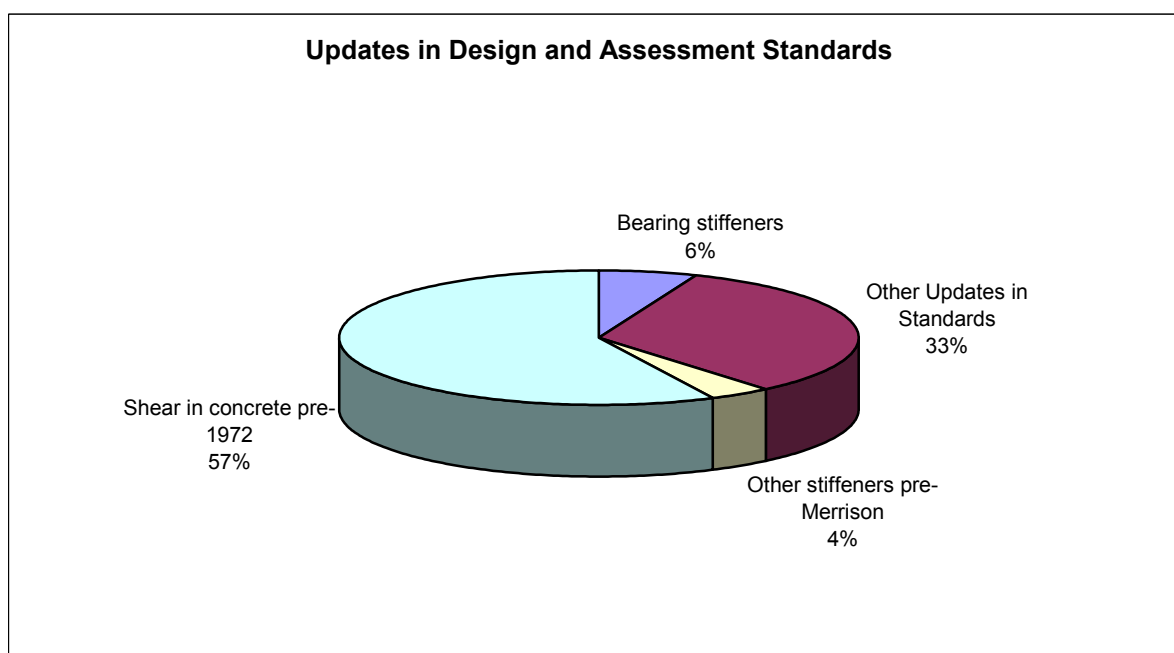


Figure 6.2

6.3.1 Review of Standards

There have been significant historical changes in the methods of design of highway bridges. Research into structural behaviour has advanced, leading to developments in the design criteria for calculating structural capacity. In some cases our improved understanding of structures has led to a reduction in conservatism (often associated with empirical formulae). In other cases the research has highlighted possible failure modes that were not previously considered, leading to stricter design rules. These developments in the standards can give rise to concerns about the capacity of older structures that were designed to older standards.

6.3.1.1 Shear in Concrete

One particularly significant change in the design standards for highway bridges was the revised method for designing concrete bridges for shear, introduced in CP110 1972. The shear clauses were developed largely in response to the work of the Shear Study Group of the Institution of Structural Engineers. As well as revising the requirement for shear reinforcement, the code introduced the need for additional longitudinal reinforcement to resist shear. Reinforced concrete may sometimes be modelled using a truss analogy, with reinforcement providing the vertical and horizontal tension forces and concrete struts providing the diagonal compression forces. The requirement for additional longitudinal reinforcement recognises that longitudinal tension forces are necessary for an effective truss analogy.

In addition, CP110 required the longitudinal reinforcement to be sufficiently anchored beyond the support. In most cases this entailed providing an anchorage length of 12 bar diameters beyond the bearing centreline. There is a marked difference in the reinforcement design of concrete bridges before and after 1973 for this reason, leading to problems when the older bridges come to be assessed.

The concrete bridge assessment code BD44/95 is largely based on the design code BS5400 Part 4, which replaced CP110 and retains the requirements for longitudinal steel. BD44/95 also contains provision for assessment of bridges where the longitudinal steel is insufficient or inadequately anchored, but these clauses can lead to highly onerous reductions in shear capacity:

- If the provided anchorage length is less than 12 bar diameters then BD44/95 requires that an empirical reduction factor be applied to the concrete component of resistance.
- In addition, if the longitudinal anchorage is insufficient to provide the required tension then the shear stirrups are deemed to be totally ineffective.
- Furthermore, enhancement of shear capacity at short shear spans is not permitted unless at least 20 bar diameters of anchorage length have been provided.

The structure strength can therefore be significantly reduced due to poor anchorage of longitudinal reinforcement.

However, it is recognised that BD44/95 is on the conservative side where the longitudinal anchorage is low. PB initiated and has part funded research with EPSRC at Bath University that has led to a new method that may be particularly useful for the assessment of concrete bridges with low longitudinal anchorage. This method provides a reduction factor for the shear capacity, derived from a lower bound model, that is a function of the fully anchored capacity, and is consistent with the existing assessment method for fully anchored reinforcement. By including the effects of shear enhancement and shear reinforcement, the new method can give significantly higher shear capacities at low anchorage lengths than the existing method in BD44/95. This new method achieves excellent correlation with experimental findings, significantly better than the current BD44/95 approach.

Another requirement in BD44/95 is that the shear stirrup spacing should not exceed the effective depth, or the stirrups will be deemed ineffective. This can also have a significant effect on the assessment capacity.

The rules for enhancement of shear capacity at short shear spans have been adjusted in BD44 from the design code BS5400. Shear enhancement is permitted for assessment within shear spans of three times the effective depth instead of only twice the effective depth in the design code, and is allowed in slabs as well as beams for assessment. These rules for assessment are therefore less conservative than the corresponding design rules.

6.3.1.2 Compressive Membrane Action

Concrete deck slabs that have sufficient in-plane restraint at their boundaries may be designed or assessed using BD81/02, which allows consideration of compressive membrane action. This recent standard provides a method that allows the ultimate strength to be enhanced according to the degree of restraint provided to the boundaries. Compressive membrane action can account for a significant amount of "hidden strength" under suitable support conditions.

6.3.1.3 Pre-Merrison Steel Bridges

The rules for design of steel box girder bridges significantly changed following two bridge collapses in 1970: Cleddau Bridge in Milford Haven, UK and the Westgate bridge over the Yarra River, in Melbourne, Australia. The committee of inquiry concluded that BS153, the British Design Standard of the time, was inadequate and came up a set of Interim Design and Workmanship Rules, often

referred to as the Merrison rules. These rules, published in 1973, comprised a comprehensive design method for steel box girder bridges, and were largely incorporated into BS5400 Parts 3, 6 and 10, and the assessment standard BD56/96. Following the publication of the Merrison rules, older bridges were assessed and strengthened. Nevertheless, many steel bridges designed before 1973 (“pre-Merrison”) still appear to have too few stiffeners when assessed to BD56.

Similarly, old riveted steelwork bridges were often designed without bearing stiffeners, with only an end plate at the abutments. The eccentricity of loading on this plate can often lead to a failure, according to BD56.

6.4 Conservative or inappropriate methods of analysis

The assessment of most bridges is highly dependent on the method of analysis. This is particularly true for concrete bridge decks, since the flexural behaviour of concrete slabs at ultimate limit state is usually significantly different from the behaviour predicted by an elastic analysis.

If a conservative or inappropriate analysis is chosen, then the bridge could appear to have an unrealistically low capacity, and this can significantly affect the outcome of the assessment. In many cases bridges have failed assessments purely as a result of the choice of analysis method. Because the audit has been based upon those structures that are still reported as substandard, the true extent of the use of inappropriate analysis techniques in early assessments may have been masked by subsequent further work that has shown them to be satisfactory, and therefore has resulted in them falling outside the scope of those being audited in the present study.

It was found that in many cases, simple Level 1 assessments had not been developed any further, and the bridges had been assumed to be substandard. Other bridges had been analysed with more sophisticated models that were not appropriate, suggesting assessment failures. A breakdown of particular analysis problems is given in Figure 6.3; each of these issues is described in more detail below.

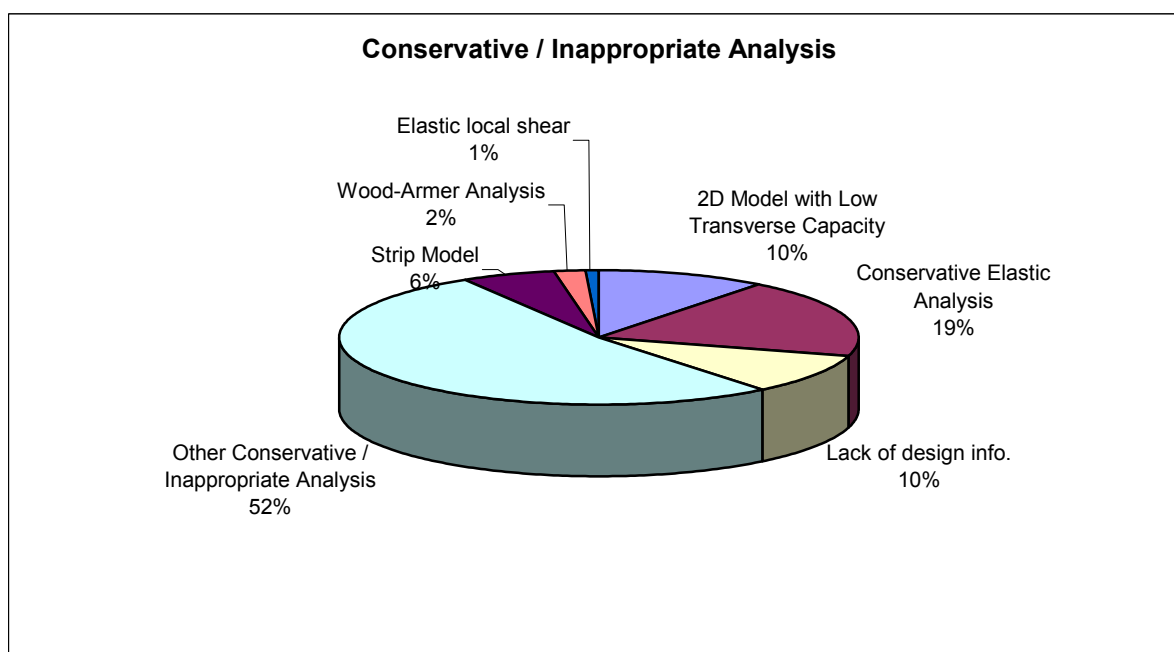


Figure 6.3

6.4.1 Conservative Elastic Analysis

The Standard BD44/95 specifies that

“Elastic methods may be used to determine the distribution of forces and deformations throughout the structure ... However, non-linear and plastic methods of analysis ... may be used with the agreement of the Overseeing Organisation.”

The use of plastic analysis can often model the ultimate behaviour of bridges far better than an elastic analysis, and so can be particularly useful for ultimate limit state assessment. However, an elastic analysis is usually performed for assessment, and the results can often be very conservative.

There are restrictions on the use of plastic redistribution analysis for concrete bridge assessment. In particular, care must be taken when dealing with shear forces. The possibility of a non-ductile shear failure means that it can often be harder to provide justification for using a plastic redistribution analysis for shear than it would be for flexure. There has been plenty of research on shear plasticity, using effectiveness factors to model the limited concrete ductility, but its application for practical assessment is relatively rare. The assessment Standard BD44/95 specifies that

“Shears and reactions used in assessment are taken as those calculated either prior to or after redistribution, whichever are the greater.”

According to this Standard, it is therefore necessary to do an elastic analysis for the shear assessment, and then, if a redistribution analysis is appropriate for flexure, both moments *and* shears should be checked after redistribution. It is possible that the shear forces in the redistributed analysis could be greater than in the elastic analysis.

An initial elastic analysis for a concrete bridge assessment that appears to fail in flexure (but passes in shear) often has the potential to be developed into a less conservative analysis that would model the ultimate behaviour more appropriately, using plastic redistribution.

6.4.2 2-D Model with Low Transverse Capacity

Some bridges were originally designed to be considerably stronger in the longitudinal direction than in the transverse direction, with the intention that at ultimate limit state the loads would be carried primarily by the provided longitudinal strength.

However, when assessed, these bridges frequently appear to be deficient in transverse strength. This result can arise simply because the model used for assessment is based on the elastic stiffnesses, which may be comparable in the longitudinal and transverse directions, rather than the ultimate strengths.

Sometimes a concrete slab might have very little transverse steel. In this case the most appropriate analysis might be to model the slab as a one-way series of longitudinal beam strips. But the analyses chosen for the assessments were frequently based on 2-dimensional grillage-type models that assumed an elastic distribution of loads. This type of analysis is likely to suggest transverse moments that exceed the low transverse capacity.

The reason that grillage analyses were chosen for these bridges might be due to a conservative interpretation of clause 6.1 of BD21/01, which states:

“In establishing the load capacity of a bridge, the effects of vehicle loading should be assessed by using some form of distribution analysis which will take advantage of the transverse distribution properties of the deck.”

If this clause is interpreted as referring to the distribution properties at ultimate limit state, it does not preclude the use of a strip analysis for slabs with low amounts of transverse reinforcement. However,

if interpreted as referring to the elastic distribution, the clause might appear to require a 2-dimensional analysis such as a grillage analysis.

For this reason, many structures failed assessments because an elastic grillage analysis indicated that the transverse strength was insufficient. If the analysis method had been more appropriate to the specific reinforcement arrangement, for example, if a yield line analysis or a basic strip analysis had been used, then the structures might have passed.

6.4.3 Strip Model

A basic beam strip model may be appropriate for a bridge deck with very little transverse strength. However, for bridges that have significant transverse distribution properties, the use of a one-way strip model could be very conservative, since any concentrated loads would be carried directly to the support with no transverse distribution, placing a high demand on the longitudinal strength of the bridge.

A simple strip model would be ideal for a Level 1 assessment in accordance with BA79/98, where a basic, conservative model is assumed to simplify the analysis and provide a quick, cost-effective result. If the basic Level 1 assessment were able to demonstrate that the bridge would pass the assessment, then no further analysis would be required. However, where the basic analysis indicates an assessment failure, the priority should be to develop the analysis to a more appropriate level of refinement to model the bridge more accurately.

6.4.4 Elastic Local Shear

Local shear effects from an elastic analysis can often seem to be critical for assessment, where a punching shear analysis would provide a more realistic model.

Peaks in shear intensity over a localised region can sometimes be suggested by an elastic analysis, particularly if a finite element analysis has been used and there are concentrated loads or discontinuities in the geometry. These peaks in shear intensity may have very high magnitudes, which may tend to infinity at discontinuities (particularly when through-thickness effects are ignored as they typically are). In reality, any failure mechanism will occur over a finite width, and so any local concentrations of shear would be better modelled using the punching shear rules in BD44/95.

A similar problem can often occur with concentrated loads close to supports, where a strip analysis has been assumed, leading to a large concentration of shear at the support. The concentration of shear is directly related to the strip width, but rigorous justification for an appropriate width may be lacking. The punching shear rules would provide a solution to this problem.

6.4.5 Wood-Armer Analysis

The Wood-Armer equations are frequently used to design reinforcement in slabs. The equations show how the bending moments and twisting moments (slab torsions) from an analysis can be combined to provide an efficient and safe way of designing the slab reinforcement. This method is convenient for design, but it is often too conservative when the equations are used in reverse for assessment. The Wood-Armer equations have been optimised for use in design to give the minimum reinforcement, but this optimisation is not relevant for assessment where the reinforcement is fixed. However, the principle behind the Wood-Armer equations can be used for assessment by employing the variation on the method as proposed by Denton and Burgoyne (this method is the basis of RASSP, which is an enhancement module for LEAP 5, developed by PB).

The use of the Wood-Armer equations for assessment can frequently lead to a low assessment capacity, where the revised method by Denton and Burgoyne would give a more appropriate rating.

6.5 Conservative assumptions

Some assessment failures appeared to be caused by particularly conservative assumptions. In some cases there might have been a lack of design information, and so in the absence of further investigations, conservative assumptions were made. This particularly applies to concrete strengths and soil parameters, but might also happen where the detailed design of a structure is not well documented, for example, it may not be clear exactly how much reinforcement has been provided.

6.6 Inappropriate use of standards

It is always possible that Standards can be misinterpreted or used for circumstances that they were not intended for. Very few of the audits fell into this category.

6.7 Deterioration in condition

Some of the audited structures had failed assessments due to poor condition. Indicators for the condition had been evaluated in an inspection, which were subsequently incorporated into the assessment calculations, either by explicitly considering the loss of section, or through the use of a condition factor.

The deterioration included the following:

- Corrosion
- Cracking
- Spalling
- Thaumasite attack
- ASR cracking
- Masonry loss
- Distortion

6.8 Design or Construction Errors

It was judged that on some occasions the reason for assessment failure was primarily due to problems with the original design or construction of the structure. The relative incidence of these problems is illustrated in Figure 6.4.

Some structures that were audited did not appear to have been designed to any Standards. Examples of these include concrete slabs with filler beams (but no other reinforcement), carrying farm access tracks.

Other structures did not appear to be based on sensible design concepts. Some of these designs indicated a possible global instability of the structure, such as a buried concrete box with no moment connections at the corners.

A few structures had no transverse reinforcement at all, while in others the reinforcement was not properly detailed. A common problem was that the verge was not designed to carry live loading, and was therefore much weaker than the carriageway part of the deck.

Substructure problems included weak pilecaps, insufficient bearings. Some of these issues appeared to be associated with the construction, although this was not always clear from the information provided in the audits.

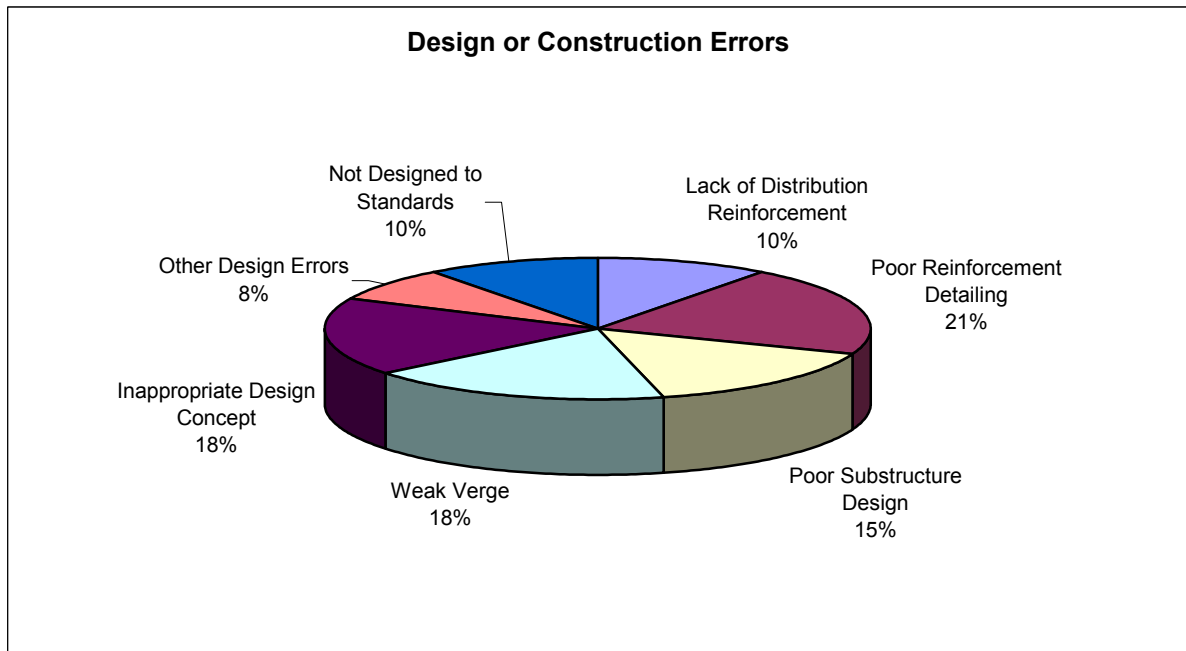


Figure 6.4

7 OPTIONS FOR FUTURE REVISIONS

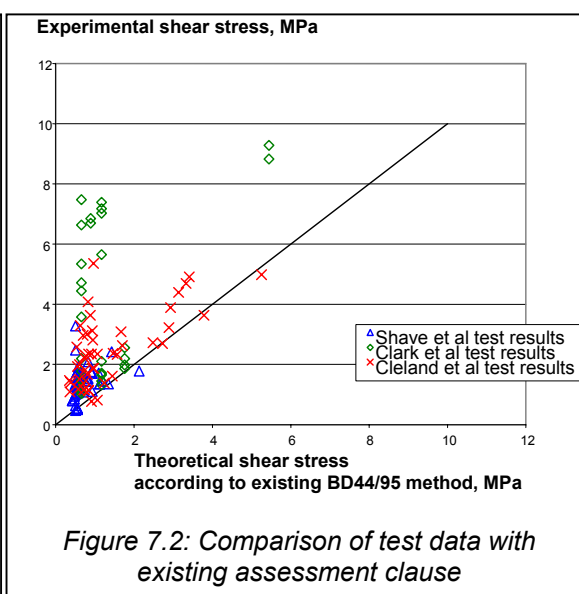
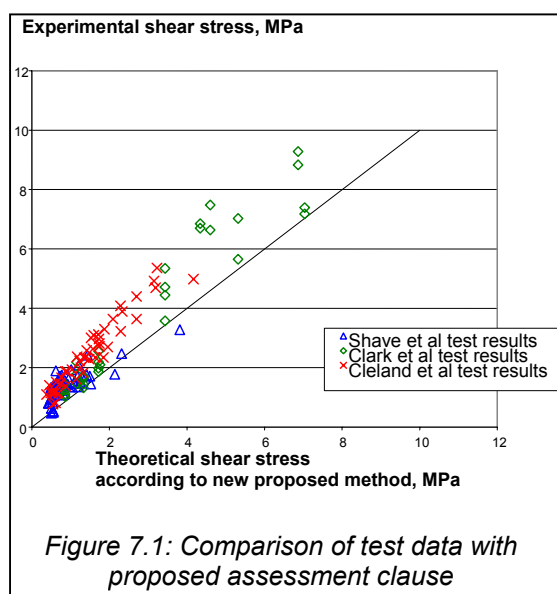
7.1 Shear in concrete: the effect of longitudinal reinforcement anchorage

Reinforced concrete bridges designed prior to 1972 frequently fail assessments according to BD44/95 due to inadequate reinforcement anchorage at the supports.

A new proposed method for assessment of these bridges has been developed at the University of Bath, partly funded by PB, with additional input from the Highways Agency. The appropriate use of this method could demonstrate the adequacy of many bridges with poorly anchored reinforcement.

The research has already been used to show that some culverts in the Highways Agency Area 2, near Bristol, have sufficient strength, and do not require strengthening. The cost of this strengthening, if it had been required, would have been in excess of £350,000. The research has therefore already led to savings several times in excess of the cost of funding the research.

The new method allows the contributions of shear enhancement and shear reinforcement to be considered, even where the longitudinal anchorage is poorly anchored. The proposed method matches the experimental data far better than the existing clause in BD44/95, as illustrated in Figures 7.1 and 7.2.



It is recommended that this new method should be considered as a possible revision to BD44/95.

The new method is presented below as a proposed revision to BD44/95 clauses 5.3.3 and 5.8.7:

Draft Proposed BD44 clauses 5.3.3 and 5.8.7
Version 6 06/11/2003 JDS

5.3.3 Shear resistance of beams

5.3.3.1 Shear stress

The shear stress v at any cross section shall be calculated from:

$$v = \frac{V}{b_w d}$$

where

V is the shear force due to ultimate loads;
 b_w is the breadth of the section, which, for a flanged beam shall be taken as the rib width;
 d is the effective depth to tension reinforcement.

In no case shall v exceed $0.92 \sqrt{\frac{f_{cu}}{\gamma_{mc}}}$ or $\frac{7}{\sqrt{\gamma_{mc}}} \text{ N/mm}^2$ (where γ_{mc} is the partial safety factor for concrete given in 4.3.3.3), whichever is the lesser, whatever shear reinforcement is provided.

5.3.3.2 Shear capacity

Shear reinforcement may take the form of vertical links, inclined links or bent-up bars, and shall only be considered effective in resisting shear if the spacing of the legs of links in the direction of the span and at the right angles to it does not exceed the effective depth, d , and if

$$A_{sv} (\sin \alpha + \cos \alpha) \frac{f_{yv}}{\gamma_{ms}} \geq 0.2 b_w s_v \quad \text{and} \quad \alpha \geq 30^\circ$$

where

A_{sv} is the cross-sectional area of shear reinforcement at a particular cross section;
 s_v is the spacing of the shear reinforcement along the member;
 α is the inclination of the shear reinforcement to the axis of the member;
 f_{yv} is the characteristic, or worst credible, strength of the shear reinforcement but not greater than 480 N/mm²;
 γ_{ms} is the material partial safety factor for steel given in 4.3.3.3.

In the absence of effective shear reinforcement, the ultimate shear resistance, V_u , of a section is given by

$$V_u = \Gamma \kappa \xi_s v_c b_w d$$

Where effective vertical links are present the ultimate shear resistance V_u of a section shall be taken as

$$V_u = \Gamma \left(\kappa \xi_s v_c b_w d + \frac{f_{yv}}{\gamma_{ms}} \frac{d}{s_v} A_{sv} \right)$$

V_u should not be taken as less than $\xi_s \frac{0.24}{\gamma_{mv}} (0.15 f_{cu})^{1/3} b_w d$.

Γ is a reduction factor to allow for inadequate anchorage of the longitudinal reinforcement, and shall be calculated as follows:

$$\Gamma = \sqrt{\frac{\alpha_s F_{ub}}{F_{ub \max}}} \quad \text{but not greater than unity.}$$

where

$$\alpha_s = 1.0 + \frac{1.6\sigma}{0.3 f_{cu}}$$

α_s is a factor to account for increased bond due to transverse pressure in the support region, but not greater than 2.6, where σ is the transverse pressure on the reinforcement due to ultimate loads.

F_{ub} is the total ultimate anchorage force in the tension reinforcement at the front face of the support as calculated in 5.8.6.3, but $\alpha_s F_{ub}$ should not be taken as greater than $A_s f_y / \gamma_{ms}$ and

$$F_{ub \max} = 6 \xi_s v_c b_w d$$

In the above expressions

$$\xi_s = \left(\frac{500}{d} \right)^{1/4} \quad \text{but not less than } 0.7;$$

v_c is the ultimate shear stress in concrete and shall be calculated as follows:

$$v_c = \frac{0.24}{\gamma_{mv}} \left(\frac{100 A_s}{b_w d} \right)^{1/3} (f_{cu})^{1/3}$$

In the formula for v_c the term $(100 A_s / b_w d)$ should not be taken less than 0.15 or greater than 3.0.

κ is a factor for shear enhancement near supports and is calculated as in 5.3.3.3.

γ_{mv} is a partial safety factor that is taken as 1.25 if the characteristic concrete strength is used and 1.15 if the worst credible strength is used.

The term A_s is that area of longitudinal tension reinforcement that continues at least a distance equal to the effective depth, d , beyond the section being considered. However, at supports the full area of tensile reinforcement may be used provided it continues to the support. Where both top and bottom reinforcement is provided, the area A_s used shall be that which is in tension under the loading that produces the shear force being considered.

Sections within a distance d from the support need not be assessed for shear providing the shear reinforcement calculated for the section at distance d is continued up to the support.

Inclined links or bent-up bars shall be assumed to form the tension members of one of more single systems of lattice girders in which the concrete forms the compression members. The maximum stress in any link or bar shall be taken as f_{yv} / γ_{ms} .

Bent-up bars shall be checked for anchorage (see 5.8.6.3) and bearing (see 5.8.6.8).

5.3.3.3 Enhanced shear strength of sections close to supports

If the main reinforcement continues to the support then an enhancement of shear strength may be allowed for sections within a distance $a_v < 3d$ from the face of a support, front edge of a rigid bearing or centre line of a flexible bearing.

This enhancement shall take the form of an increase in the allowable concrete shear stress $\xi_s v_c$ by a factor $\kappa = (3d/a_v)$, but the

total shear stress v should not exceed $0.92 \sqrt{\frac{f_{cu}}{\gamma_{mc}}}$ or $7 / \sqrt{\gamma_{mc}}$ N/mm², whichever is the lesser.

5.3.3.4 Bottom loaded beams

Where load is applied near the bottom of a section, sufficient vertical reinforcement to carry the load to the top of the section shall be present in addition to any reinforcement required to resist shear.

5.8.7 Curtailment and anchorage of reinforcement

Curtailment lengths and anchorages of bars shall be assessed by rigorous analysis at the curtailment or anchorage point for the worst load case in accordance with 5.8.6.3.

5.8.6.3 Anchorage bond To prevent bond failure the tension or compression in any bar at any section due to ultimate loads shall be developed on each side of the section by an appropriate embedment length or other end anchorage. The anchorage bond stress, assumed to be constant over the effective anchorage length, taken as the force in the bar divided by the product of the effective anchorage length and the effective perimeter of the bar or group of bars (see 5.8.6.4), shall not exceed the value

$$\beta \sqrt{f_{cu}} / \gamma_{mb}$$

where:

β is a coefficient dependent on bar type, and given in Table 15A;
 f_{cu} is the characteristic, or worst credible concrete cube strength;
 γ_{mb} is a partial safety factor equal to 1.4, unless the worst credible stress is used, in which case it is equal to 1.25.

Bar type	β	
	Bars in tension	Bars in compression
Plain bars	0.39	0.49
Type 1: deformed bars	0.56	0.70
Type 2: deformed bars	0.70	0.88
Fabric	0.91	1.13

Table 15A Values of bond coefficient β

7.2 Distribution Analysis

There are a large number of concrete slabs failing assessments due to inadequate transverse flexural capacity. However, some of these apparent failures may be caused by an analysis method that is not appropriate for the design of the bridge. If there is very little transverse reinforcement then a grillage or finite element analysis will inevitably suggest a transverse flexural failure.

BD21/01 clause 6.1 states the following:

“In establishing the load capacity of a bridge, the effects of vehicle loading should be assessed by using some form of distribution analysis which will take advantage of the transverse distribution properties of the deck.”

This statement is a little ambiguous regarding the meaning of *transverse distribution properties*. It has often been interpreted as referring to the transverse elastic stiffness properties of the structure, so that in practice it may have been assumed that a 2-dimensional elastic grillage or finite element model is required to simulate the distribution.

But the ultimate limit state load distribution is determined not by the elastic stiffnesses but by the ultimate strengths associated with the provided reinforcement arrangement. For example, if there is little or no transverse distribution steel in a slab, then it could be said that the slab has little or no *transverse distribution properties* at the ultimate limit state, and so it would be inappropriate to use a complete 2-D distribution analysis such as a grillage. This analysis would be attempting to take advantage of transverse distribution properties that do not exist in the structure.

Clarification of this clause would result in more appropriate assessment of slabs with low amounts of transverse reinforcement.

7.3 Levels of Assessment

The levels of assessment given in BA79/98 provide guidance on refining an assessment using increasingly sophisticated analyses. In the event of an assessment failure at Level 1, then a refined structural model is used for Level 2. If this fails then bridge-specific material strengths and loads are considered for Level 3. Levels 4 and 5 are rarely used, but involve reliability analysis.

However, the sequential order of these levels may not always be appropriate for particular bridges. Taking cores to establish worst credible material strengths can be a quick way of showing that a structure is adequate, and is often more efficient than analysing the structure with a more complicated structural model. Application of Level 3 could therefore be an appropriate step between Levels 1 and 2.

Furthermore, a simple model may be the most appropriate structural analysis for some structures, and a more refined model could have misleading results, as discussed in Section 6. Clarification of these assessment levels might reduce the total cost of the assessment process.

7.4 Condition Factors

The condition factor described in BD21/01 clauses 3.18-3.19 does not appear to provide a rational method for the ultimate limit state assessment of many structures.

It is recommended that any reduction in capacity due to condition should be appropriate to specific mechanisms of failure, and should model the effect of the condition at each part of the structure on the ultimate behaviour of that part of the structure.

Global condition factors should not generally be encouraged, since poor condition can change the critical ultimate failure mechanism for a structure, which would not be predicted by the use of a global condition factor. For example, concrete cracking may have a negligible effect on the ultimate flexural capacity, although shear cracking can reduce the concrete's ability to carry shear by aggregate interlock. The presence of shear cracking could possibly suggest that shear would become the critical mechanism.

Clear guidance has been provided for the use of condition factors for arch assessment. However, the general application of condition factors with arbitrary reasoning for any structure can lead to inappropriate assessment.

7.5 Enhancing the Understanding of Assessing Engineers

The assessment of existing structures is a complex problem that presents different challenges and demand different expertise from design. Fundamentally, it requires a deeper understanding of structural theory and in particular a recognition of the true way that structures behave as the ultimate limit state is approached.

In contrast, in design, provided an analysis method is used that gives a distribution of stresses throughout the structure in equilibrium with the applied loads which are used as the basis for determining the required section capacities, the resulting design will be safe (according to the lower bound theory of plasticity). Further consideration of the actual behaviour at ULS, which may be quite different, is neither required nor typically merited.

Enhancing the understanding of assessing engineers is not straightforward, and neither is seeking procurement routes that successfully differentiate between assessing organisations with the appropriate level of technical expertise and those without it. Notwithstanding the difficulties, opportunities to address these challenges should be embraced at a policy level.

One approach that does offer a potentially effective means of strengthening the level of technical discipline brought to the assessment process is through the technical approval process. The provision of enhanced guidance to those responsible for technical approval, setting out important

aspects of achieving realistic assessments could be beneficial in this respect. For example, the guidance included in Appendix A relating to the realistic assessment of concrete slabs in flexure (which was one of the most common types of assessment failure identified) could be suitable for wider dissemination and could be complemented with companion guidance on other assessment-related issues.

8 CONCLUSIONS AND RECOMMENDATIONS

A sample of 294 structures has been audited in order to appraise the results of the assessment element of the Assessment and Strengthening programme.

The audits have provided data identifying the following:

- the relative incidence of each type of structure failing assessment
- critical elements, locations and potential modes of failure
- the reasons for assessment failures.

On the basis of these results and other research, the following specific recommendations have been made, and are described in greater detail in Section 7:

- BD44/95 clauses 5.3.3 and 5.8.7 should be updated to deal with shear in bridges with poorly anchored reinforcement, as described in Section 7.1
- BD21/01 clause 6.1 should be clarified to encourage appropriate analysis of structures with little distribution reinforcement.
- The Assessment levels in BA79/98 should be clarified to encourage the possibility of material testing following a Level 1 assessment failure, when appropriate.
- BD21/01 clauses 3.18-3.19 should be clarified to prevent inappropriate use of the condition factor.

In addition, it is recommended that approaches to enhance the level of technical understanding brought to bridge assessment be embraced. For example, the provision of further guidance, such as that given in Appendix A, is encouraged to support those responsible for technical approvals.

9 REFERENCES

BA16	The Assessment of Highway Bridges and Structures
BA35	Inspection and Repair of Highway Structures
BA38	Assessment of the Fatigue Life of Corroded or Damaged Reinforcing Bars
BA43	Inspection, Repair and Monitoring of Post-tensioned Concrete Bridge Decks
BA44	The Assessment of Concrete Highway Bridges and Structures
BA51	The Assessment of Concrete Structures affected by Steel Corrosion
BA54	Load Testing for Bridge Assessment
BA63	Inspection of Highway Structures
BA79	The Management of Sub-standard Highway Structures
BD2	Technical Approval of Highway Structures on Trunk Roads
BD21	The Assessment of Highway Structures
BD34	Technical Requirements for Assessment and Strengthening Programme for Highway Structures: Stage 1 – Older Short Span Bridges and Retaining Structures
BD44	The Assessment of Concrete Highway Bridges and Structures
BD46	Technical Requirements for Assessment and Strengthening Programme for Highway Structures: Stage 2 – Modern Short Span Bridges
BD48	The Assessment and Strengthening of Highway Bridge Supports
BD50	Technical Requirements for Assessment and Strengthening Programme for Highway Structures: Stage 3 – Long Span Bridges
BD52	The Design of Highway Bridge Parapets
BD56	The Assessment of Steel Highway Bridges and Structures
BD61	The Assessment of Composite Highway Bridges and Structures
BD63	Inspection of Highway Structures
BD81	Use of Compressive Membrane Action in Bridge Decks
BS5400	Steel, Concrete and Composite Bridges
EC2	Design of Concrete Structures

Superseded Standards

BS153	Specification for Steel Girder Bridges
BE1/77	Technical Memorandum (bridges) BE1/77: Standard Highway Loadings
CP110	Code of Practice for the Structural Use of Concrete
CP114	The Structural Use of Reinforced Concrete in Buildings

Papers and Reports

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PE Regan	"Tests of delaminated anchorages subject to transverse pressure" University of Westminster, March 1999
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APPENDIX A - FURTHER GUIDANCE FOR ASSESSMENT

Concrete Slabs in Flexure

Elasticity and plasticity

The assessment of concrete slabs can often lead to an apparent failure where an elastic analysis has been carried out. Due to the ability of concrete slabs to redistribute bending moments significantly prior to the formation of a failure mechanism, an initial elastic analysis is rarely sufficient to prove that the slab is deficient in flexure.

Simply supported beams are examples of *statically determinate* structures, which may be analysed using equilibrium alone, and which will form a simple collapse mechanism when one part of the structure reaches its capacity. Continuous structures and slabs, on the other hand, are examples of *statically indeterminate* structures. The elastic analysis of these structures is affected by stiffness properties, and the inherent structural redundancy provides robustness. A collapse mechanism is not automatically formed as soon as one part of the structure reaches its capacity, and it is necessary for other parts of the structure to be involved in the mechanism.

When parts of the structure start to yield or behave non-linearly, the rest of the structure begins to attract a larger proportion of the load, until eventually a failure mechanism will form. If the structure is ductile, the full plastic capacity will be developed simultaneously at all points in the failure mechanism. This assumption underpins the application of plasticity theory. Ductility is therefore necessary to justify the use of plasticity theory to predict plastic redistribution.

Concrete slabs are usually ductile in flexure, unless they are either over-reinforced, have no reinforcement in one direction or are deteriorated. Plasticity can usually be justified to redistribute loads for a flexural assessment of a concrete slab.

Redistribution of moments can cause the shear forces to increase as well as decrease from their elastic levels. It is therefore necessary to check shears as well as moments when using plastic redistribution. However, shear failures can be brittle and so it is also necessary to check shears using an elastic analysis.

Examples where transverse redistribution is not used

There are instances where an elastic analysis may be the best solution even for a concrete slab, and there is no benefit in further analysis. For example, a simply supported one-way spanning slab that has distributed loads that are uniform across the full width will already be resisting those loads uniformly across the full width (behaving like a wide beam), and no transverse redistribution of loads is possible. In this situation an elastic analysis would be the best solution, and the slab could even be modelled as a simple line-beam. But if the loads were concentrated (for example, wheel loads) then the elastic solution would predict peaks in the bending resistance corresponding to each load. These peaks may be reduced by allowing the slab to redistribute the bending moments plastically in the transverse direction.

However, there is a limit to how much redistribution is possible. If the total flexural capacity across the full width of the slab is insufficient to sustain the bending moment when calculated on the full width, then there is no point carrying out any redistribution analysis. The full-width calculation corresponds to the maximum possible lateral redistribution and is therefore a useful preliminary check since if this fails then it is clear that redistribution will not work. This calculation corresponds to an upper bound analysis with a full width yield-line mechanism.

Conversely, if the full-width check is successful this does not necessarily imply that the capacity is adequate, since it is essential that there is sufficient transverse capacity to allow the lateral distribution; a more detailed analysis is required.

If there is no transverse reinforcement at all then the transverse bending capacity will be very low (limited by concrete tensile strength, which is usually neglected) and the structure will often have negligible ability to transversely distribute the load. There would be no benefit, for example, in performing a grillage or finite element analysis because the transverse moments will inevitably be too large. In this situation a longitudinal strip analysis may be the only option, with the width of the strip based on load dispersal at 45 degrees through the depth of the slab to the neutral axis.

The 45-degree dispersal method appears to be conservative since a real failure mechanism would usually involve a greater width of slab. However, it is difficult to quantify the realistic critical width

when there is no transverse reinforcement. A yield-line analysis would appear to predict a critical mechanism as shown in Figure 2 when optimised for zero transverse bending capacity. However, this model predicts no transverse bending behaviour, since the transverse hogging and sagging yield-lines become aligned in the optimisation, so that the mechanism is equivalent to a transverse shear mechanism with vertical shear cracks. However, this is not a realistic mechanism, since a shear failure would always occur with an inclined shear crack. Hence the effective width of the longitudinal strip could be estimated by assuming that the transverse failure can be modelled by a diagonal shear crack, inclined at a particular angle. Using the rules for punching shear this would indicate a strip width of $1.5d$ on each side of the load, leading to a minimum strip width of $3d$ (plus the load width).

It appears doubtful whether the punching shear rules are appropriate with no reinforcement (although this is not mentioned in BD44/95), and it is not suggested that the resistance due to transverse shear should be included in the strip calculation, however this crude approach provides a useful estimate of the minimum strip width.

Plasticity theory

Concrete slabs generally have sufficient ductility in flexure (assuming they are in good condition and have suitable reinforcement in both directions) to allow use of plasticity theory in the form of the upper-bound and lower-bound theorems. Both of these simple theorems can be very useful, when applied appropriately, in demonstrating that a bridge has sufficient capacity after an initial elastic analysis indicates a failure.

Yield-line analysis of slabs is a very common and powerful application of upper-bound theory. The method is called upper-bound because the calculated capacity for each particular mechanism is higher than or equal to the true capacity. However, if all possible mechanisms are considered then the correct capacity may be obtained. The method itself can be very simple to apply (by comparing the work done by loads with the energy dissipated in the structure), but it can give an unsafe result unless the mechanisms are appropriately optimised to give the worst capacity. It is therefore essential to ensure that all possible mechanisms have been considered (including fan mechanisms). Simple structures may be analysed using a program such as SYLAS, developed by PB, or COBRAS, developed by Dr Campbell Middleton at Cambridge University.

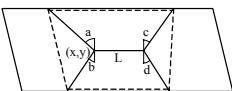
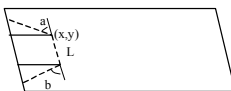
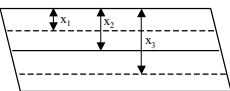
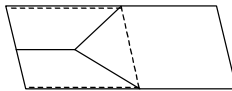

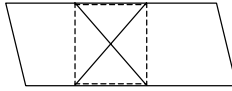



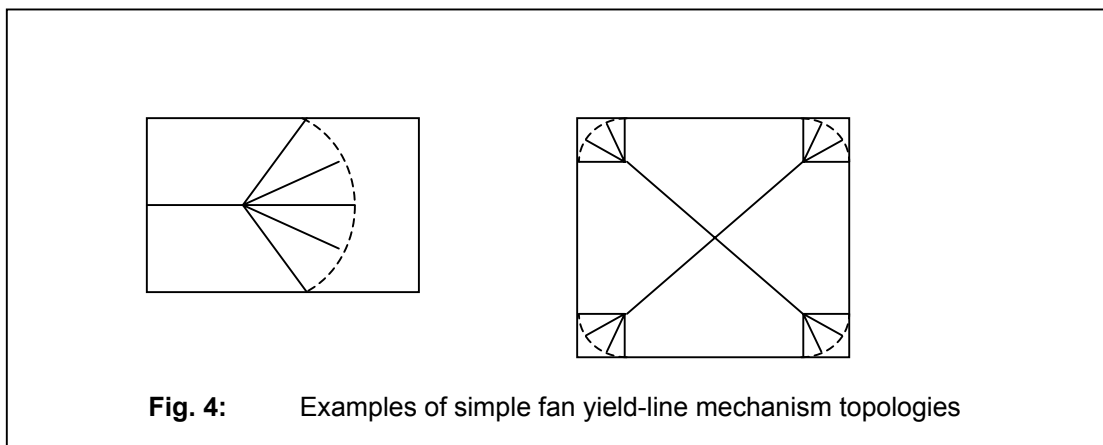
	Mechanism 1	Mechanism 3	Mechanism 4
General topology			
Typical special cases	  	 	 <div data-bbox="1029 1765 1300 1892"> <p>KEY</p> <p>— Sagging yield-line</p> <p>--- Hogging yield-line</p> </div>

Fig 3: SYLAS yield-line mechanism topologies



For a structure with complicated geometry or a complex reinforcement layout it may be difficult to predict the overall topology of potentially critical failure mechanisms with confidence. This prediction of potentially critical topologies is the essentially initial step before they are optimised to give a critical mechanism. In such cases, yield-line analysis may not be the most efficient or appropriate method.

Edge beams of concrete slabs have been observed to separate in a brittle fashion, and it is good practice to neglect them in yield-line analysis. This may be done by ignoring any loads applied to the edge beams as well as any strength due to the edge beams. The main slab may then be treated as if the edge beams have already separated. However, the edge beams should also be separately assessed for the loads applied directly to them.

Lower-bound theory is generally less well understood, although it is effectively used for the majority of designs and assessments. Lower-bound theory simply requires any system of internal stress resultants (e.g. bending moments) that are in equilibrium with the loads to be within their section capacities. The analysis provides a lower bound on the true structural strength, i.e. the method is safe. An elastic grillage analysis followed by a check of internal member capacities is therefore one particular application of the lower-bound theorem.

However, the lower-bound theorem applies to any equilibrium system, even if the stiffnesses used in the analysis are varied. It can be useful to adjust the relative stiffnesses in an analysis, since this will change the way the loads are distributed and can often provide a less conservative result. Hence if an initial slab analysis indicates a transverse failure but no longitudinal failure, the relative stiffnesses can be adjusted, reducing the transverse stiffness. This will reduce the transverse moments but increase the longitudinal moments, and can sometimes show that the structure is adequate. Often the use of cracked section properties rather than full section properties will have a similar result if there is less transverse steel than longitudinal steel. BD44/95 specifically allows the use of cracked section properties for elastic analyses, and also allows non-linear and plastic analyses, provided they are agreed with the Overseeing Organisation.

Some grillage analysis programs are able to place a limit on the bending moments, so that the program effectively inserts plastic hinges wherever the moments reach their limit. This feature is a useful application of lower-bound theory, but is often unavailable for grillage analysis (it is possible in LEAP 5 classic).

Lower-bound methods are often used for the analysis of statically indeterminate beams (e.g. continuous beams or beams that are built-in at their supports). The redundancy in the structure may be exploited to allow plastic hinges to be formed in structure without causing collapse. One method of achieving this is to consider the total bending moment diagram to be the superposition of the free bending diagram and the reactant diagrams, with each reactant diagram corresponding to a degree of redundancy in the structure (without any loads applied). The free bending diagram corresponds to the response of structure to the loads with all redundancies removed, for example a simply supported beam. The coefficients associated with each reactant diagram may then be adjusted so that the total moments are within plastic limits. This approach is illustrated in Figure 5(a).

Alternatively, it is equivalent to consider the superposition of the normal elastic solution and the bending diagrams due to imposing rotations at points in the structure, as illustrated in Figure 5(b). This approach can be simply applied to slabs as well as beams, and has been developed by PB for

assessment of bridge decks. The slab may be analysed elastically with a torsionless grillage, and then, if the moments exceed the bending capacity in a localised region, this method may be used to redistribute moments out of that critical region. A moment release may be inserted at the most critical position, and a rotation imposed. The bending moments in the structure due to this action may then be superposed with the original elastic solution. The coefficient associated with the moments due to the rotation may be adjusted in order to bring the maximum total moments in the structure within their plastic limits. If necessary this process may be repeated with hinges in other positions. This can be a quick and effective way of demonstrating that a structure is adequate for flexural assessment.

This method of lower bound optimisation of assessments may be carried out for slabs using the flowchart in Figure 6.

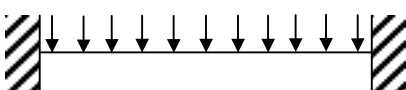
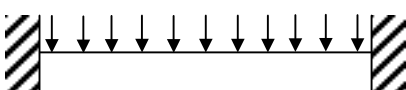
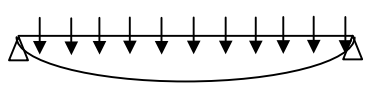
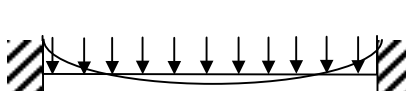
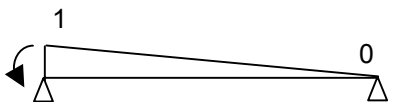
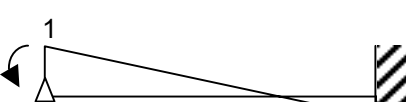

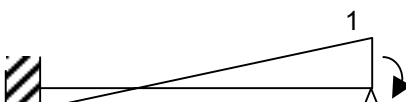
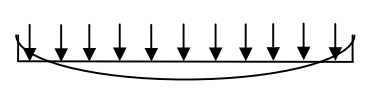
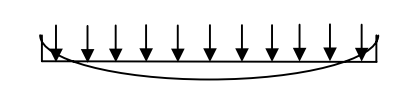
			
M_{FB}		$M_{Elastic}$	
β_1		β_{Rot1}	
β_2		β_{Rot2}	
M_{Total}		M_{Total}	
	$M_{Total} = M_{FB} + \sum \alpha_i \beta_i$ <p>Coefficients α_i to be adjusted to bring M_{total} within capacity limits</p> <p>(a) Free bending plus reactant diagrams</p>		$M_{Total} = M_{Elastic} + \sum \alpha_i \beta_{Roti}$ <p>Coefficients α_i to be adjusted to bring M_{total} within capacity limits</p> <p>(b) Elastic plus rotation diagrams</p>

Fig 5: Illustration of 2 possible applications of lower bound plasticity theory for an encastre beam. Total bending moment diagram M_{total} may be calculated as the weighted superposition of the diagrams shown. (a) Free bending plus reactant diagrams method, (b) Elastic plus rotation diagrams method. The method shown in (b) may be simply adapted for grillage analysis of slabs.

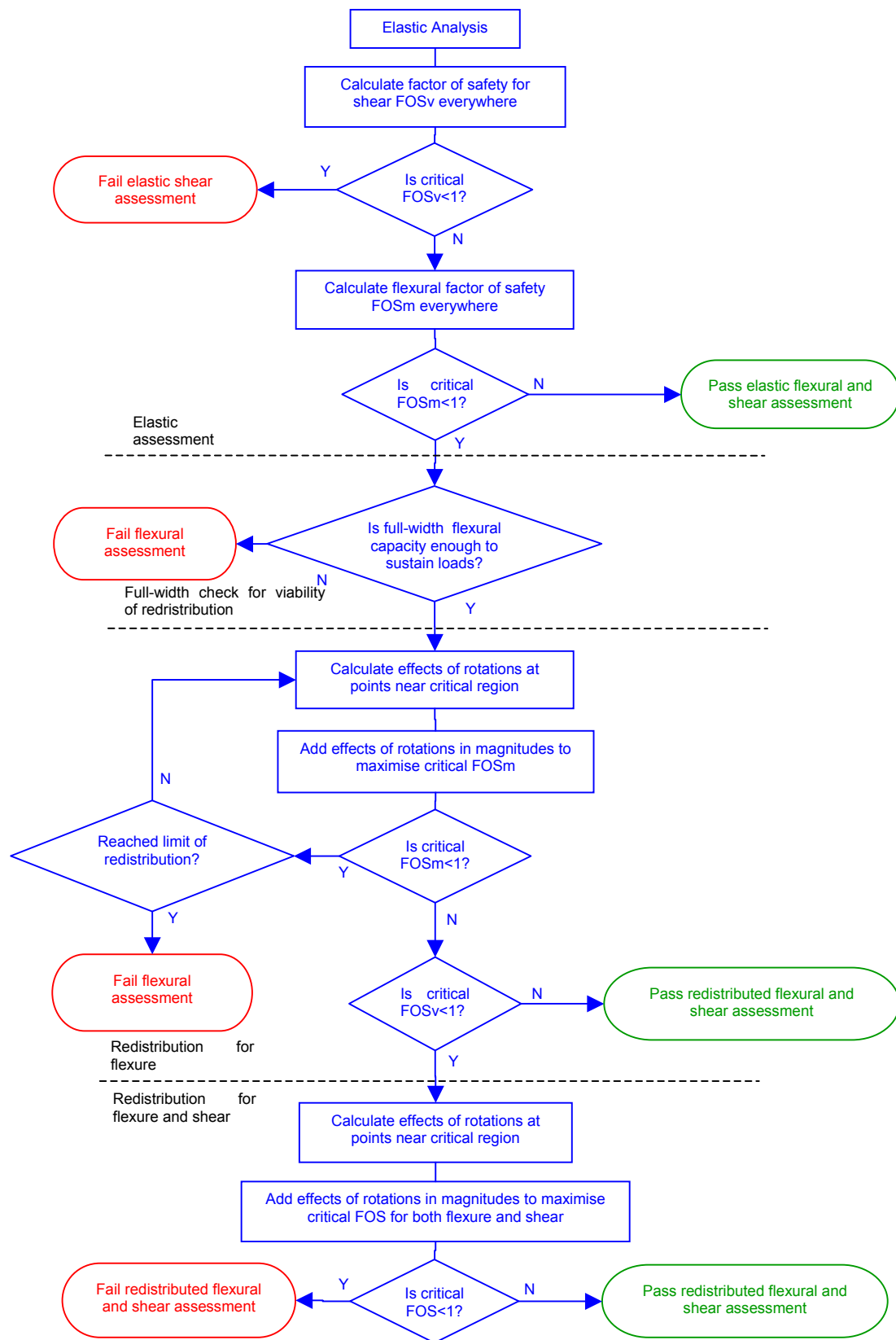


Fig 6: Flowchart for optimised lower bound slab assessment using redistribution

Bending and twisting in slabs

The Wood-Armer equations are frequently used to design reinforcement in slabs. The equations show how the bending moments and twisting moments (slab torsions) from an analysis can be combined to provide an efficient and safe way of designing the slab reinforcement. This method is convenient for design, but it is often too conservative when the equations are used in reverse for assessment. The Wood-Armer equations have been optimised for use in design to give the minimum reinforcement, but this optimisation is not relevant for assessment where the reinforcement is fixed. However, the principle behind the Wood-Armer equations can be used for assessment by employing the variation on the method as proposed by Denton and Burgoyne (ref 28). This method is the basis of RASSP, which is a bolt-on program for LEAP, developed by PB.

A distinction should be made between torsion in individual elements (eg beams) and twisting moments in slabs. Torsion failures are frequently brittle, and so the torsional strength of beams is usually neglected in yield-line analysis (e.g. where a yield-line crosses a beam at a skew angle). Similarly, it is questionable whether lower-bound redistribution of beam torsions is valid. However, twisting moments in slabs are rather different, even though in a grillage program they will appear as if they are beam torsions. Twisting moments are observed where the reference axes differ from the principal axes of bending, as illustrated in Figure 7. If the arbitrary reference axes were chosen to align with the principle axes of the bending moment field at one position, then the twisting moments about the new axes would decrease to zero at that point. The twisting moments are therefore flexural resultants rather than true torsions, and since flexure in concrete slabs is ductile under normal conditions, it is valid to use plasticity with twisting moments.

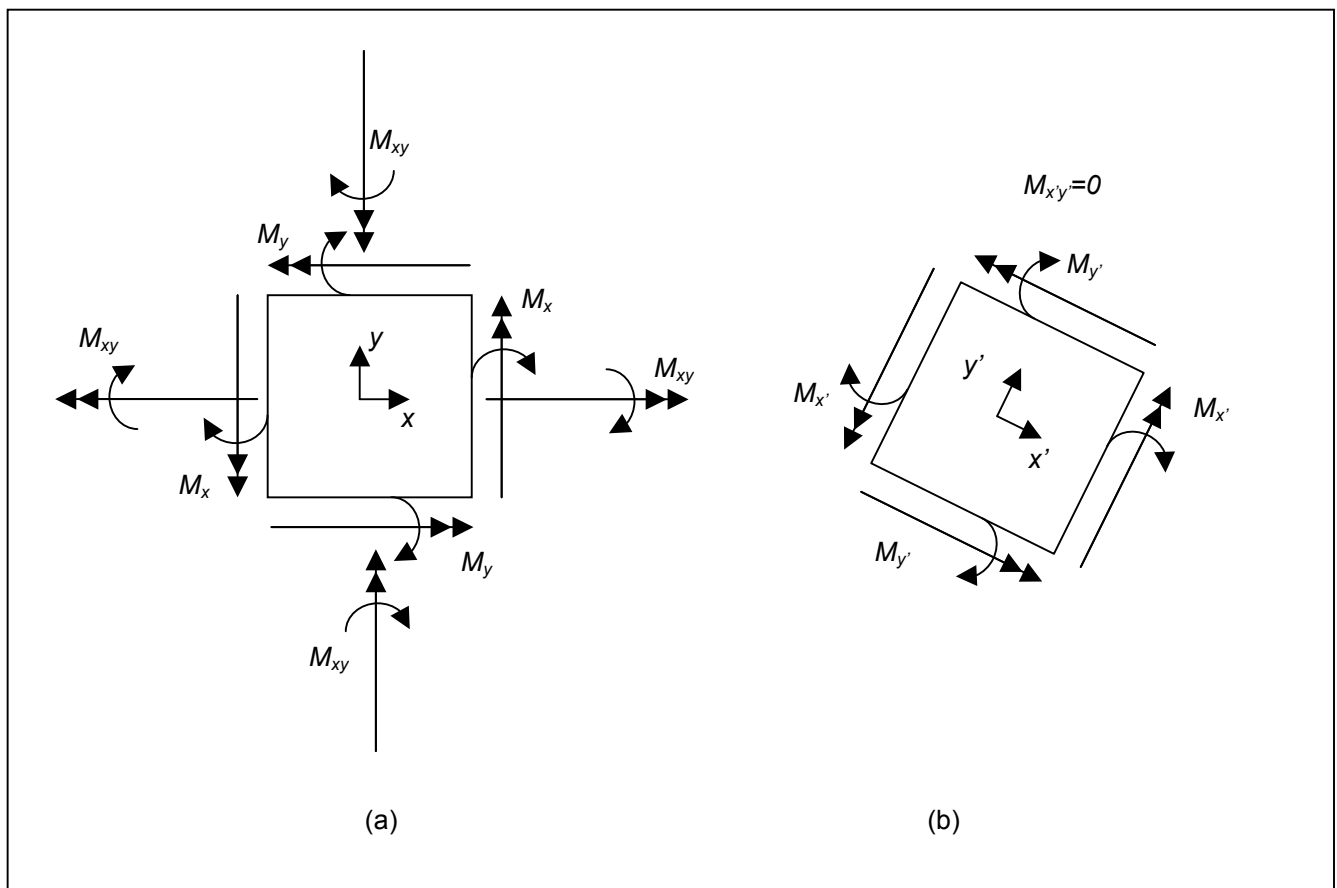


Fig 7: Bending moments and twisting moments. (a) general axes (x, y) , (b) specific axes (x', y') aligned with the principal axes of bending so that $M_{x'y'}=0$. Note that some programs may use different definitions and sign conventions to the senses illustrated.

A common use of the lower-bound theory for slab analysis is to neglect the torsional stiffness of grillage elements, redistributing the loads by forcing the principal bending moments in the slab to align with the grillage members. This is sometimes done with a skewed grillage, so that when interpreting the results for slab assessment the principal bending moments appear to be non-orthogonal. In the real slab this would be impossible, since principal axes are always orthogonal. Although this appears to be a valid equilibrium solution according to the lower bound theory, it is not generally recommended for skew angles greater than 15 degrees.

Flowchart

Figure 8 shows some procedures for utilising some of the redistributed strength in slabs after the failure of a Level 1 assessment. The assumptions behind plasticity theory may not be appropriate for structures in poor condition.

