12 - 30 COPY No. 1

FATIGUE OF CABLES IN CABLE-STAYED BRIDGES

FINAL REPORT

Prepared for

National Cooperative Highway Research Program
Transportation Research Board
National Research Council

acer/FREEMAN FOX

In association with :-

TWI THE WELDING INSTITUTE

and

MORRISON-KNUDSEN ENGINEERS, INC.
A MORRISON KNUDSEN COMPANY

APRIL 1991

ACKNOWLEDGEMENT

This work was sponsored by the American Association of State Highway and Transportation Officials, in cooperation with the Federal Highway Administration, and was conducted in the National Cooperative Highway Research Program which is administered by the Transportation Research Board of the National Research Council.

DISCLAIMER

This copy is an uncorrected draft as submitted by the research agency. A decision concerning acceptance by the Transportation Research Board and publication in the regular NCHRP series will not be made until a complete technical review has been made and discussed with the researchers. The opinion and conclusions expressed or implied in the report are those of the research agency. They are not necessarily those of the Tranportation Research Board, the National Research Council, or the Federal Highway Administration, American Association of State Highway and Transportation Officials, or of the individual states participating in the National Cooperative Highway Research Program.

TABLE OF CONTENTS

FINDINGS CONTINUED

LIST OF FIGURES

LIST OF TABLES

ACKNOWLEGEMENTS

The research reported herein was performed under NCHRP Project 12-30 by Freeman Fox Ltd.(now Acer Freeman Fox), Consulting Engineers in association with The Welding Institute (now TWI) and Morrison-Knudsen Engineers Inc. (Consulting Engineers).

The work was performed under the general supervision of C. Walter Brown, Director, Acer Freeman Fox.

Jolyon A. Gill, Senior Engineer, Acer Freeman Fox, was the principal investigator. Dr. Martin H. Ogle, Principal Design Consultant,(TWI) was co-principal investigator. Cade L. Benson, now Operations Manager, Morrison-Knudsen Engineers Inc. (Denver) was an investigator.

ABSTRACT

This report documents the work carried out during NCHRP Project 12-30 which covers the fatigue of cables in cable-stayed bridges. The objectives of the project were:

(i) to develop criteria and guidelines for fatigue design of cable stays, and

(ii) to develop practical guidelines for material requirements and for testing wires, strands and cable stays.

Existing data has been collected from many sources including research reports, papers and articles as well as directly from stay cable manufacturers and bridge owners and designers. These data were analysed to produce the following information:

(A) A summary of fatigue tests on large steel wire cables, modified where necessary to indicate a consistent failure criteria. A lower bound (mean minus two standard deviations) is proposed as a design line.

(B) A comparison between:-

(a) fatigue damage implicitly adopted when the methods of the present AASHTO Specification are used, and

(b) fatigue damage derived from a model of real truck traffic flow for cable-stayed bridges (400 ft - 1500ft main spans).

(iii) Recommended Design Stress Ranges to be adopted for fatigue design of cable-stays when using the methods of the present AASHTO Specification.

(iv) Recommendations for material requirements and for testing of cable stays.

(v) The recommendations in (iii) and (iv) above are incorporated into a Draft Guide Specification for the fatigue design of cables in cable-stayed bridges.

(vi) Comments on some design aspects are included as well as recommendations for future research work in this field.

SUMMARY OF FINDINGS

The objectives of the project were met with the production of a Draft Guide Specification for the fatigue design of all current forms of steel wire cables in cable-stayed bridges. The body of the report acts as a commentary on this Draft Guide Specification.

The project commenced with a literature search and the issue of questionnaires to owners and designers of existing cable-stayed bridges in order to elicit fundamental information which would form a data base of knowledge.

As the project progressed it became clear that the depth and quality of data anticipated was not going to be forthcoming. Nevertheless it was possible to address both the loading and the resistance sides of the basic fatigue design equation.

The report is innovative in its highlighting of the major fatigue damage effect created by bunches of trucks on long span bridges. It was possible to maintain for longer spans, the same design approach as that adopted by the AASHTO Specification for shorter spans. The loading/resistance equation was carefully addressed and the resistance side was modified to allow for the fact that the application of the single fatigue load in the AASHTO Specification is not realistic for longer span bridges.

The Draft Guide Specification is also innovative in adopting a performance based approach to acceptance of stay cable systems. This ensures that the relatively young cable-stay industry will have the freedom to develop and prove new and efficient systems for use on the cable-satyed bridges of the future whilst at the same time being able to rely on the background of the well tried and tested systems that exist at present.

Several very practical issues are raised. The report strongly recommends and the Draft Guide Specification stipulates that stays must demonstrate their ability to resist corrosion throughout their design lifetime. The stays must also be easy to inspect and, in case it is necessary, they shall be economically replaceable without disruption to the traffic. In addition, all aerodynamic effects shall be minimised through good design practice and the use of dampers if necessary.

It is reported that considerably more information is necessary and further research required to develop the data bases on traffic behaviour and endurance performance of cable-stays to a state where more confidence could possibly lead to relaxation of some of the more conservative requirements in the fatigue design of cables in cable-stayed bridges.

CHAPTER 1

INTRODUCTION AND RESEARCH APPROACH

1.1 PROBLEM STATEMENT

In the last 30 years cable-stayed bridges have become an increasingly advantageous and economical type of structure for medium- and long-span crossings. This report addresses the flexible steel cable elements of cable-stayed bridges. There is a variety of proprietary designs available on the market at the present time, but there are few specifications that adequately address these systems at the moment. The steel cable stays are vital components and are normally designed to carry a high dead load tensile stress together with significant positive and negative stress fluctuations due to live load, wind load and local bending. Fatigue is thus a potentially important design consideration. As yet, no design codes contain comprehensive rules to give satisfactory long term cable performance. However some criteria have been provided in the German Code DIN 1073 (Ref 1) and considerable guidance is given in the Post-Tensioning Institute publication of January 1986 entitled "Recommendations for Stay Cable Design and Testing" (Ref 2). No explicit guidance is given in the AASHTO code (Ref. 3).

However, considerable basic research has been carried out worldwide, and the results are presented in various reports. This research together with experience from actual structures can be used to provide a basis from which design criteria and material requirements can be developed to suit US practice. This project set out to achieve that aim.

1.2 RESEARCH OBJECTIVES AND SCOPE

The objectives of this project were:

(i) to develop criteria and guidelines for fatigue design of steel cable-stays, and

(ii) to produce practical guidelines for material requirements and for testing wires, strands and cable stays were to be produced.

In order to achieve these objectives the project was divided into a total of seven tasks. Each task is briefly described below:-

1.2.1 Task 1.

A review of performance history and data, current domestic and foreign codes of practice, and research findings was undertaken. This information was assembled from both technical literature and unpublished experiences of designers and owners of cable-stayed bridges. Although this desk study concentrated on fatigue behaviour in cables of cable-stayed bridges, relevant aspects of fatigue in other structural systems was considered. For example, owners and designers of some suspension bridges were contacted where their experiences with suspenders was likely to be relevant.

1.2.2 Task 2.

Analysis and evaluation of the information generated in Task 1 was carried out to establish rationales for approaches to the development of design criteria and testing requirements for fatigue effects in cables. This evaluation included consideration of the following:

(i) The loading to be used in fatigue appraisal.
Lane loading and individual truck loading were considered to determine which was most appropriate for US specifications. Intensity and frequency was analysed including the effects of overtaking or passing vehicles. The implications of structural configuration, (ie. spacing and arrangement of stays) was also assessed.

(ii) The effect of wind, including quasi-static and aeroelastic excitation, both on the structure and on the stays themselves.

(iii) The effects of local stresses in stays from causes other than wind. These included effects from clamps, saddles and sockets.

(iv) The assessment of fatigue strength of cables from tests on short lengths.

(v) Suitable methods of cumulative damage assessment.

(vi) Maximisation of fatigue resistance through careful attention to quality control and assurance of wire and strand.

1.2.3 Task 3.

The findings of Tasks 1 and 2 were presented in an interim report. This included examples illustrating the approaches considered.

1.2.4 Task 4.

Cable fatigue design provisions were prepared in a format suitable for consideration by the AASHTO Subcommittee on Bridges and Structures and incorporated into a Draft Guide Specification. The main text of the report acted as a commentary and a design example was given to facilitate understanding and use of the design provisions.

1.2.5 Task 5.

Materials and testing requirements were drafted and included in the Draft Guide Specification. These were based on performance criteria.

1.2.6 Task 6.

Additional research needed for further development and refinement of the recommended design criteria and materials requirements was identified.

1.2.7 Task 7.

The Project was completed with the preparation of a final report.

1.3 GLOSSARY AND DEFINITIONS

Commonly accepted definitions have been used, in general, throughout this Project. However, by way of clarification, some terms together with their definitions as used in this report are listed below:-

Stay -	The complete tension member including anchorages, main tension elements, sheathing and all corrosion protection materials, clamping and damping devices and saddles, all anchored into a local area of superstructure.

Cable - The complete stress carrying metallic section of the main tension components which are anchored with a single socket. This does not include any sheathing, blocking agents, clamps, sockets or anchorage components. (See Fig. 1)

Bundle - A group of open or close packed parallel cables, held together by clamps or wrapping so forming a single stay.

Wire - The smallest single tension component in all cables except for parallel bar cables. Usually circular in cross section with a diameter between 1/8" (approx. 3mm) and 5/16" (approx. 8mm) but may be non circular as in locked coil strands. Capable of being coiled for transportation.

Strand - A small number of wires laid helically around a central straight wire not necessarily in the same direction. Seven-wire and 19-wire strands are the most common. The lay length is generally long so that the inclination of the external wires in relation to the central wire is small, so ensuring a relatively high modulus of elasticity. Capable of being coiled for transportation.

Spiral bridge strand - A large number of wires laid helically around a central straight wire in usually more than 3 layers. The direction of lay of the helix is often alternated between layers of wires.

Locked coil strand - (sometimes referred to as locked coil rope)	Similar to a spiral bridge strand except that the outer layers of wires (and possibly some internal layers) are formed with non-circular wires so shaped that they interlock with each other, leaving virtually no voids in the cross-section.
Long lay spiral bridge stand -	Strand similar to a spiral bridge strand except that the wires have a longer lay such that, at first sight, they may appear to be nearly parallel to the axis of the cable.
Steel Bar -	The smallest single tension component in a parallel bar cable, usually circular in cross-section and usually exceeding 0.5" (13 mm) in diameter. It is supplied in short straight lengths connected with intermediate sockets (usually threaded) and not capable of being coiled for transportation.

[The basic cross sections of some cables are given in Fig.1.]

Rope - A group of strands laid helically round a central strand. Rope is not normally used for cable-stays because of its low and uncertain modulus of elasticity.

Blocking Agent - The material used to fill (or partially fill) the interstices within and around the cable to keep moisture out and to provide lubrication. Materials such as oil, grease, wax or cement grout may be used. Some may be introduced at the time of laying up the cable, others may be introduced under pressure after the cable has been completed.

Sheathing - Metal or plastic covering used to enclose the cable and provide permanent protection against corrosion or damage. This may be in the form of a pipe designed to resist pressure during injection of blocking agents.

Tape - A continuous flexible adhesively backed tape of polyvinyl fluoride (PVF) or similar material which is wound round a stay or sheath to provide a water, ultra violet (UV) and heat proof barrier.

Cable Clamp - A strap tightened round a bundle of cables at intervals to maintain a defined cross-sectional shape.

Anchorage - A device comprising all components and materials such as sockets, bearing plates, pins, threaded rods etc. required to retain the force in a cable and to transmit this force to the structure of the bridge deck, tower, or foundations.

Socket - A permanent enclosure at the end of a cable to enable stress to be transferred from the cable to the rest of the anchorage. Stress transfer between the cable and socket is usually by cast alloy metal, epoxy resin, wedge grips, threaded nuts, button heading or a combination of these. The socket is usually a permanent fixture on the cable but may be adjustable under load relative to the rest of the anchorage.

Wedge Grips - Tapered metal components for transferring the tension by friction from a wire, strand or bar into a socket or other anchorage component.

Button Head - A cold formed upset end to a wire that enables the tension to be transferred to a socket or other anchorage component in bearing.

Socket Filler Material - A material introduced whilst liquid into a socket to surround the individual cable elements and which then solidifies or sets hard to provide a structural bond between cable and socket.

Saddles - A curved trough-shaped structural component that supports one or more cables and permits them to change direction without excessive bearing stresses. Note that saddles may be used to transfer a proportion of cable tension to the structure. In this case the saddle may be considered as a type of anchorage.

Splay Saddle -	A saddle which enables individual cables in a bundle to diverge to separate anchorage points. It may nor may not be supported in contact with the structure.
Damper -	A device clamped to a stay to absorb vibration energy. May be freely attached inertia type or viscous type fixed between stay and structure.
Truck	In the context of this report, a vehicle unit with a gross weight of 5 kips or more.
Design Life	The required endurance, usually in years, during which the component is required to perform with the minimum specified degree of damage.

Fatigue life -	The endurance expressed as the number of cycles to a defined failure condition under a given constant amplitude stress range.
Fatigue Limit -	The constant amplitude stress range below which fatigue life is infinite. (Note that this report does not recommend the use of a fatigue limit.)
Carriageway -	That part of a highway consisting of the traffic lanes travelling in one direction. Hence a highway normally has two carriageways.
S-N curve	A graph of fatigue test results (usually having logarithmic scales) where stress range (or stress range divided by ultimate stress) is plotted on the vertical axis (S), and endurance is plotted on the horizontal axis (N).

	The endurance is expressed as the number of cycles to reach a specified degree of damage. This has been commonly taken in this report as rupture of 5% of metallic cross section and not necessarily used by other researchers.
Relative Damage	The damage caused by the number of trucks under consideration expressed in terms of UTD.
UTD	Unit Truck Damage - Damage caused by a single HS20 truck.

Stay arrangements (see Fig 2)

Single Stay	A configuration using a single cable-stay in back- span and main-span side of the tower.
Harp	A multi-stay configuration with stays arranged parallel to each other.

Fan	A multi-stay configuration with stays not parallel to each other; arranged at a spacing down the tower closer than that adopted for a harp arrangement.
Radiating	A multi-stay configuration with stays radiating from a single point at the top of the towers.
Star	An arrangement of stays anchored at single points in the deck on either side of the towers, but spaced out down the tower.

Cable-stays can be arranged either in a single plane (one set of stays at the bridge centreline) or in multi-plane arrangements (one set, either vertical or inclined, located along each bridge edge girder)

UNITS

US customary units are used in this report, with metric units given in parentheses.

NOTATION

Notation conforms to the AASHTO Specification (Ref.3).

CHAPTER 2

FINDINGS

2.1 QUESTIONNAIRES

Figure 10 illustrates the numbers of cable-stayed bridges in existence with their construction dates.

The time and budget available within this project clearly did not allow visits to such a large number of cable-stayed bridges. Thus, in order to obtain information from owners and designers, a detailed questionnaire was drawn up.

This questionnaire was produced in three parts, the first being very simple so as to present a minimal psychological barrier to a speedy response.

NCHRP comments on the questionnaires were incorporated into the final versions.

Despite considerable difficulty in obtaining names and addresses of some foreign bridge owners, 70 questionnaires were issued. Out of these, 32 replies were received with varying depths of response.

Of the 21 questionnaires issued for German bridges, it was unfortunately only possible to obtain information from two privately owned bridges because of the restrictions on release of this type of information.

Release of information from Japan was equally unforthcoming on Japanese bridges but the reasons were less well defined.

The response from the United States was good and visits were made to the owners of four out of the six listed bridges that were, at that time, open to traffic.

A list of bridges where significant information was obtained is presented in Appendix B.

The two main reasons for issuing the questionnaires were, firstly to obtain design details (such as materials, dimensions, loading and workmanship specificaton) for the structures and, secondly to discover whether any fatigue effects in the cables had been reported and if so, to relate these to the stress spectra induced in the cables by the traffic flow. In the first case, most major bridges have been the subject of publications and articles which often give some details of the structure. A certain amount of information was thus gathered without the return of questionnaires. In the second case, it was very difficult to produce any reliable figures on the cable stress spectra because detailed information on the traffic patterns had not been recorded, in general, by the owners.

It would thus seem that even if a fuller response had been made to the questionnaires their usefulness would not have been as great as had been hoped. However, information on detailed design drawings not published in the literature and on the engineer's specification was particularly useful. It should be noted that further refinement of the fatigue design provisions produced in this report would be possible if actual continuous measurements of stress spectra in cable-stays were made on as many cable-stayed bridges as possible (particularly in the US) together with a full record of vehicle weights and spacing.

A simplified data base on cable-stayed bridges has been produced in Appendix A. This gives key points on almost 200 bridges and has been assembled from questionnaire responses and from the literature that has been collected (as noted in 2.1.3 below). This part of the project will, we believe, prove valuable in its own right. No similar work, to our knowledge, has been undertaken since the paper presented in 1977 in the Proceedings of the ASCE, Journal of the Structural Division "Bibliography and Data on Cable-Stayed Bridges" (Ref.4).

Further valuable information on the present state of repair of many cable-stayed bridges was obtained from a privately conducted world-wide condition survey of the stay cables on almost half of the cable-stayed bridges listed in Appendix A. The survey was unpublished at the time but the results were made available in confidence to the research agency. The survey produced photographs giving qualitative (rather than quantitative) data on the performance of the stay cables and provided a very useful background "feel" for the sort of problems that can occur.

2.2 RESEARCH REPORTS, PAPERS, ARTICLES AND TRAFFIC SURVEYS, CURRENT US AND FOREIGN STANDARDS AND CODES OF PRACTICE

Over 200 documents relevant to this project were collected from many sources including worldwide data bases. An alphabetical list by author is presented in Appendix C.

One or more key words have been allocated to each item to facilitate listing under subject. A copy of the key-word directory is included in Appendix C.

For each of the most important references covering the field of fatigue strength a synopsis has been produced, in order to provide an overview of the type of material utilised during the preparation of this report. These synopses are presented in Appendix C.

Although by no means exhaustive, the data base of literature gives a very comprehensive background of information for this project.

A note of caution, however, should be recorded at this point. Goodyear (Ref.5) mentions that current research is revealing heavier truck traffic. Documented records of this increasing weight of truck traffic were not available in a form that could be used in this project at the time this report was being prepared. This fact is indicative of the general lack of information on current trends in traffic flow patterns on bridges in the medium-to long-span ranges in the States.

There is general information on traffic flow aimed at pavement design and static design of structures. But information on the mix of trucks and cars and on the distribution of axle weights with time throughout the day is sparse. There is very little information on patterns of flow, particularly for heavy vehicles, related to headway and congestion. For the meantime we recommend that a conservative approach to the design fatigue loading should be adopted in view of the necessarily long term view that must be considered when designing against fatigue effects.

Many codes of practice and standards have been assembled as referred to on drawings and in specifications; some of these are listed below. An attempt has been made to form a collection that is relevant to the United States and for this reason few foreign codes and standards are shown.

BRIDGE DESIGN STANDARDS

AASHTO (US)	Standard Specifications for Highway Bridges. 13th Ed. 1983 - Together with Interim Specifications of 1984 and 1985.
British Standards (UK)	BS 5400 - Steel, Concrete and Composite Bridges. - Particularly Part 10 : 1980 - Code of practice for fatigue.
German Standard (West Germany)	DIN 1073 - Steel Road Bridges - Basis of Design.
Post Tensioning Institute (US)	"Recommendations for Stay Cable Design and Testing" JANUARY 1986.

WIRE PRODUCT SPECIFICATIONS

(US)	ASTM A416-80	"Uncoated seven-wire stress-relieved strand for prestressed concrete".
	ASTM A421-80	"Uncoated stress-relieved wire for prestressed concrete".
	ASTM A586-81	"Zinc-coated parallel and helical steel wire structural strand".
	ASTM A603-70(-80)	"Zinc-coated steel structural wire rope".
	ASTM A722-75	"Uncoated high-strength steel bar for prestressing concrete".

2.3 OTHER SOURCES OF INFORMATION

Discussions with manufacturers of wire ropes and prestressing strand took place including those which specifically offer cable stay systems. These include:-

BBR

British Ropes

Freyssinet

Stronghold

VSL

These contacts proved most fruitful in terms of unpublished test results and specifications.

Other areas that were also researched included the U.S. mining industry, but we found in this case that the majority of the data available concerned wire ropes which, because of their low stiffness, are not normally used in cable-stayed bridges. The mining industry uses running ropes whereas cable-stayed bridges with static cables do not need the additional property of low stiffness to permit long life in a running rope situation.

The researchers' own experience of the behaviour of cables and suspenders in suspension bridges has proved to be useful, especially when the particular problem of cable corrosion was considered. The performance of suspenders can be likened in many respects to that of cables in cable-stayed bridges.

2.4 TRAFFIC LOADING

The objective was to anticipate the cable stress history over the design life of the structure. AASHTO does not define the assumed design life but in discussion with experts associated with the design clauses , 80 years was suggested. This is primarily dependent on the magnitude, frequency and sequence of loading. This contrasts with the AASHTO Standard Specification (Ref. 3) which only considers magnitude and frequency. The reason for the importance of sequence (or traffic pattern) is that for long spans the magnitude is very much affected by more than one vehicle on the span at any one time. This has dictated the type of traffic data necessary for this study.

The information gathered in Task 1 of this project provided data, making it possible to determine the representative spectrum of truck weights within the total truck traffic population that is typically carried on routes in the United States.

(Note that the term "truck" is used to cover vehicles weighing in excess of 5 kips, see definitions)

Mean truck flow per day was listed, as well as truck flows at hourly intervals throughout the 24 hour day. This, together with some very limited data on truck flow patterns, permitted some estimates to be made for the expected mean headway of US truck traffic.

From these two pieces of information a typical truck train was drawn up and used to determine representative cable loading histories on computer models of various types of cable-stayed bridge.

The stages in derivation of the typical truck train are described in the following parts of this section. Two later sections cover the various types of cable-stayed bridge and the use of the computer to derive cable-stay stress histories for the various types of bridge.

2.4.1 Analysis of Distribution of truck weights

The literature shows some variation in distribution of gross truck weight between the different classes of route. The FHWA Report RD-85/012 (Ref. 6) provides data from 1980-1982 which can be summarised as follows: -

	Mean kips.	Standard Deviation
Interstate roads	44	23
US routes	44	27
State routes	40	26
Mean for all routes	43	23

TABLE 2.4.1 TRUCK WEIGHT DISTRIBUTION ON ROUTES

It should be noted that data for the FHWA Report (Ref. 6) was collected from a total of 32 sites. Of these, 18 sites were Interstate roads, 6 were US routes and 8 were State routes.

In our calculations we have adopted the distribution obtained when all 32 sites are combined. This gives a mean (over the 27, 513 trucks seen at 32 sites) of 43 kips and a standard deviation of 23 kips. The truck gross weight histogram for all sites and all States is illustrated in Fig 3.

Two points should be considered at this stage: -

(i) As noted above, this distribution could be an underestimate of real truck distribution on some routes because it does not include the effects of heavier truck traffic that is currently being recorded.

(ii) The method of truck weight recording used by the researchers effectively turned real bridges into weigh bridges. An impact effect was noted but automatically smoothed out. Theoretically an impact factor should be added to each load in the train of trucks. However, it is demonstrated later that the majority of damage is inflicted by bunches of trucks and not individual trucks. In this case, it is considered that the timed occurrences of the additional impact effects is extremely unlikely to be phased such that all impact effects are additive in their global action. In other words, any overall increase in the weight of any bunch of trucks will be expected to be very small, if existent at all. In the absence of evidence to the contrary, no additional impact factor on the truck loads has been included in the calculations that follow.

2.4.2 Analysis of Truck flow and mean headway

Estimates for a realistic mean peak hourly truck headway have been derived from two sources. Each method produced a figure of 400 ft. as a reasonable, but possibly conservative, figure.

Firstly, truck flow records given in the FHWA Report RD-85/012 (Ref. 6) were taken and the implied mean headway was deduced.

Secondly, published data in FHWA Report OH-83/001 (Ref. 7) was analysed to produce a mean headway figure. These two approaches are discussed in more detail below.

2.4.2.1 Analysis of Truck flow data

The greatest mean daily truck flow recorded in the FHWA National Survey (Ref. 6) is 500 trucks per carriageway per day (State of California). However, the peak hourly flow recorded at some sites was as much as six times the mean daily flow rate (California SR 17 S. Bound - Fruitvale Av.). This can cause bunching which leads to cycles of stress due to more than one vehicle being on the structure at any one time, particularly where the relevant influence line is long.

When considering fatigue effects it must be remembered that the damaging effect on life is proportional to the stress range of each cycle of loading raised to the power of "m". In figure 9 a value of m = 3.76 has been deduced from available experimental results. (The derivation of this figure is described in section 2.10 of this report.)

It can thus be seen that if 2 trucks cross the longitudinal influence line for the cable tension together, then the damage accrued during the peak flow periods will be $2^{3.76} \div 2 = 6.8$ greater than that occurring if they cross separately and will tend dominate the fatigue damage on the cable-stays. It should be remembered when comparing with the AASHTO Standard Specification (Ref. 3), that, in that document, the possibility of more than one truck on a structure is not considered since it is not intended for design of spans of more than 500ft in length. The concept of multiple groupings of trucks occurring at peak flow periods and creating the majority of the fatigue damage, is thus a very important issue.

We have taken the mean daily truck flow of 500 trucks per carriageway per day and multiplied it by the factor of 6 (noted earlier) in order to derive the mean peak hourly truck flow rate. This gives 125 trucks per hour per carriageway. We have multiplied this by a factor of 2 to cater for several unknown effects such as:-

(i) the weight of cars adding to truck weights (9 kips of cars - say 3 cars - travelling with each average 43 kip truck will produce this factor)

(ii) local conditions that may accentuate the traffic surges such as:-

nearby junctions,

traffic lights,

toll collection,

steep gradients, etc.

A further factor of 2.5 has been included to allow for growth of truck traffic during the design life of the structure (2.25% per year for 80 years). This gives an effective heavy flow of 10 trucks per minute per carriageway.

At a mean truck speed of 45 mph this flow gives a mean headway of just under 400 ft during the period of heavy flow.. (Note that slower speeds could increase bunching effects. Note also that increased flow has been assumed to produce a greater number of separate bunches and that the concentration of load within each bunch remains the same).

2.4.2.2 Analysis of Published Headway Data

The literature contains very little published data on headway patterns occuring on the various classes of US highway.

The most useful information available was contained in FHWA Report OH-83/001. (Ref 7). Here headway records are presented for four separate locations on 2-lane single carriageways. The Average Daily Truck Traffic (ADTT) levels at these locations is not given. A total of 632 events were recorded by noting the time between trucks. Details are given of 269 events (43% of the total) where the recorded headway is less than 12.5 seconds. For the remaining 57% of events details are not given, except that they all had headways greater than 12.5 seconds.

The 269 events for which details were available were analysed and assumed a mean truck speed of 55 mph. The analysis can be summarised for the following location of trucks:-

1. Driving following driving lane: where one truck is following another down the driving lane.

2. Passing approaching driving lane: where one truck is about to overtake a truck in the driving lane.

3. Driving approaching passing lane: where one truck is about to overtake a truck in the passing lane.

4. Passing following passing lane: where one truck is following another down the passing lane. -

Location of Trucks	Total Events	Headway less than 12.5 seconds	
		Number of events	Mean Headway ft. (55 m.p.h.)
1. Driving following Driving lane	364	111	536
2. Passing approaching Driving lane	73	35	249
3. Driving approaching Passing lane	165	100	356
4. Passing following Passing lane	30	23	340
TOTAL:	632	269 (=43%)	

TABLE 2.4.2 TRUCK HEADWAY DATA

The mean headway for the 43% of the trucks most tightly bunched is 415 feet.

Several points should be noted when considering the relevance of this truck headway data: -

(i) The calculation is based on 269 events at four separate locations.

(ii) The ADTT at these locations is not known.

(iii) The sites may not be typical of traffic patterns throughout the States.

(iv) The total sample taken was 632 events of which 57% all had headways in excess of 1000 ft. Hence the mean headway over all events would be considerably greater than 400 ft.

Taking into account the above reservations we consider that, for the purposes of damage calculations for 2 lane carriageways, a mean headway of 400 feet, as used in our calculations, is an appropriate and safe figure.

2.4.3 Derivation of a typical truck train

With the information described in the previous two sections covering distribution of gross truck weights and truck flow and mean headway, it was then posible to derive a typical train of trucks. (This is illustrated in Fig 4).

The length of the train was chosen to contain a large enough number of trucks to accurately model the distribution of gross truck weights produced in the FHWA Report RD-85/012 (Ref. 6) (illustsrated in Fig 3). The weight of a total of 27,513 trucks is recorded in that report. The number of trucks in each band of weights is as follows:-

Weight range kips	Number of Trucks in FHWA Report
0 - 10	1370
10 - 20	3210
20 - 30	5838
30 - 40	3848
40 - 50	2648
50 - 60	2639
60 - 70	3570
70 - 80	2819
80 - 90	1187
90 - 100	269
100 - 110	68
110 - 120	19
120 - 130	9
130 - 140	6
140 - 150	16
Total	27,513 trucks

TABLE 2.4.3A DISTRIBUTION OF GROSS TRUCK WEIGHTS

In order to reduce the number of trucks required for the model train of trucks the above distribution of trucks was modified slightly while still maintaining the same damaging effect of the spectrum of truck weights. The number of trucks in the last three weight ranges were converted to equivalent numbers of trucks in the range 110-120 kips as follows:-

9 trucks with a mean weight of 125 kips create the same damage as 9 x $(125/115)^{3.76}$, ie 12 trucks, with a mean weight of 115 kips.

Similarly 6 trucks with a mean weight of 135 kips create the same damage as 11 trucks with a mean weight of 115 kips

and 16 trucks with a mean weight of 145 kips create the same damage as 38 trucks with a mean weight of 115 kips.

In this way it was possible to delete the last three weight ranges from the above spectrum and to add a total of (12 + 11 + 38) = 61 trucks to the 110 - 120 kips range to give a total, for this range, of 80 trucks.

The number of trucks for the computer simulated truck train was then chosen by allocating one truck to each of the two highest ranges in the modified spectrum. Proportionally higher numbers of trucks were allocated to each of the remaining ranges depending on the numbers in each range of the spectrum. This gave a total of 372 trucks and is detailed in the table below (the percentage error on the truly theoretical number of trucks is given in the last column and the error on the fatigue damage is a 0.5% overestimate):-

Weight range kips A	Modified number of trucks from the FHWA report B	Number of trucks in truck train used in computer simulation C	Percentage error in number of trucks in truck train D
0 - 10	1370	18	- 3%
10 - 20	3210	43	- 1%
20 - 30	5838	79	0
30 - 40	3848	52	0
40 - 50	2648	36	+ 1%
50 - 60	2639	36	+ 1%
60 - 70	3570	48	- 1%
70 - 80	2819	38	0
80 - 90	1187	16	0
90 - 100	269	4	+ 10%
100 - 110	68	1	+ 9%
110 - 120	80	1	- 7%
	TOTAL	372 trucks	error on fatigue damage + 0.5%

TABLE 2.4.3B MODIFIED DISTRIBUTION OF GROSS TRUCK WEIGHTS

An in-house computer program was especially extended for this project to derive trains of trucks by selecting sequence and headway in a random fashion from the given input data on the spectrum of truck weights and the mean headway. This programme then proceeded to run its chosen train of trucks across defined structures and to calculate the relative damage incurred in a given number of defined members (cables).

This section of the report covers the way the typical truck train was derived.

The spectrum of truck weights, as defined in columns A and C of the table above, was entered into the computer program. The computer then chose, at random, a sequence of trucks from this spectrum using each truck only once. At the same time a headway was allocated to each truck as it was chosen so that a train of 372 trucks was built up. The allocated headway was based on the mean headway of 400 ft. but chosen at random between zero and twice this value. (Headways of less than normal running distance between trucks therefore represent overtaking trucks).

In this way it was possible to generate any number of trains of trucks each containing 372 trucks each with the same total spectrum of truck weights and each with a mean headway of 400 ft.

In order to simplify later computations, it was necessary to select one particular train of trucks as being representative. To this end the computer was used to generate a series of truck trains and each train was run across one cable-stayed bridge model in order to record the relative damage incurred by the cables. Choosing one particular cable, the variation in relative damage was plotted for each train of truck generated. After a total of 22 truck trains had been generated and run across the model cable-stayed bridge, the distribution of relative damage had stabilised. It was then possible to select the truck train that produced the mean relative damage and use this particular train as the design truck train for all remaining computation.

Figure 4 shows this design truck train used in our calculations. (As noted above, where headways are less than the normal running distance between trucks, this signifies that overtaking is occurring.)

2.5 BRIDGE GEOMETRY

Having derived a train of trucks, computer models of cable stayed bridges were produced over which this train could be run in order to calculate cable stress histories.

2.5.1 Span Range

The cable-stayed bridge with the longest main span, at present, is Annacis Bridge in Vancouver, Canada, with a span of 1526 ft (465m), opened in September 1986. At the other end of the scale, cable-stayed construction has been chosen for very short spans, but usually for non structural reasons. The bridge data base in Appendix A shows that the typical minimum main span for cable-stayed highway bridges is about 400 ft.

Accordingly the range of computer models has been kept to structures with main spans between 400 ft and 1500 ft.

2.5.2 Other structural parameters

The arrangement of cable-stays has a considerable bearing on the length of influence line obtained. So also does the method of backspan support (e.g. multi-piers etc.). Similarly the relationship between deck stiffness and cable area (and hence cable stiffness) is important. Deck and towers can be of concrete or steel and the support conditions and pier locations can vary considerably.

Computer models of 10 cable-stayed bridges were produced. They were chosen to incorporate most of the above variations in structural parameters. These are described in more detail in the following section.

2.5.3 Computer Models

SITKA

Single stay, two planes, with composite deck. Main span 450 ft, two side spans of 150 ft each.

HAWKSHAW

Single stay, two planes, with steel deck. Main span

713 ft, two side spans of 189 ft each.

WYE

Single stay, single plane, with steel deck. Main span 770 ft, two side spans of 285 ft each.

ERSKINE

Single stay, single plane, with steel deck. Main span 1000 ft, two side spans of 360 ft each.

PASCO KENNEWICK

Radiating stays, two planes, with concrete deck. Main span 981 ft, two side spans of 407 ft each.

LULING

Fan formation of stays, two planes, with steel deck. Main span 1222 ft, two side spans of 508 ft and 495 ft.

JAMES RIVER

Harp formation of stays, single plane, with concrete deck. Main span 630 ft, two 300 ft side spans with intermediate piers at 150 ft.

DAME POINT

Harp formation of stays, two planes with concrete deck. Main span 1300 ft, two side spans of 650 ft each.

CHAO PHYA

Fan formation of stays, single plane with steel deck. Main span 1476 ft, wide spans with two intermediate piers at 201 ft.

ANNACIS

Fan formation of stays, two planes with composite deck. Main span 1526 ft, two side spans of 600 ft each.

In addition to the 10 models of cable-stayed bridges described above, two families of imaginary single stay cable- stayed bridges were also produced in order to study, in more detail, any possible span effect.

The first family consisted of extrapolations from the designs for Sitka and Hawkshaw and produced a family of four bridges with spans of 450ft. 600ft., 713ft. and 900ft.

The second family consisted of extrapolations from the designs for Wye and Erskine and produced a family of four bridges with spans of 500ft., 770ft., 973ft. and 1250 ft.

2.6 STAY-CABLE STRESS HISTORY

Having obtained a load train (Section 2.4) and a series of computer models (Section 2.5) a load history in several stay cables was produced for each bridge considered by stepping the load train across each computer model. This was performed with the aid of the computer program specially adapted for this project.

A measure of the resulting fatigue damage was obtained by the program and this was compared with an equivalent figure obtained from the passage of a single HS20 truck across each bridge model. It was thus possible to express the fatigue damage caused by the passage of the train of trucks in each cable in terms of Unit Truck

Damage (UTD). The figures are summarised in the table below and show that, on average, the design truck train produces damage which is equivalent to the passage of 200 HS20 trucks. The range of damage varies from 130 to 270 UTD.

The calculation of the relative fatigue damage was performed by the program as follows:-

The load ranges in each cable were counted using the reservoir method described in many references such as BS 5400 (Ref. 8). This gave the number and magnitude of effective load ranges experienced by the stay-cable during the passage of the train of trucks.

The fatigue damage in a cable is proportional to the stress (and hence load) in the cable raised to the power of the slope of the S-N curve (as described in section 2.4.2 above). The magnitude of each load range was therefore raised (by the program) to the power of 3.76 (the slope of the S-N curve dervied in section 2.10 of this report) and then multiplied by the number of occurrences of this range. These figures were then summed for each cable to give a figure which was proportional to the relative damage inflicted upon the cable. A similar operation was performed for the passage of the single HS20 truck and the two results compared and expressed in UTD as summarised in the table below.

TABLE 2.6 RELATIVE FATIGUE DAMAGE EFFECT

CAUSED BY THE DESIGN TRAIN OF 372 TRUCKS

ON COMPUTER MODELS OF VARIOUS BRIDGES

BRIDGE	MAIN SPAN ft.	CABLE NO. REF.	RELATIVE DAMAGE. [UNITY = DAMAGE BY ONE HS 20 TRUCK]
SITKA	450	1 & 2	204
HAWKSHAW	713	1 & 2	216
WYE	770	1 & 2	210
ERSKINE	1000	1 & 2	212

TABLE 2.6 (Cont'd)

BRIDGE	MAIN SPAN ft.	CABLE NO. REF.	RELATIVE DAMAGE. [UNITY = DAMAGE BY ONE HS 20 TRUCK]
PASCO-KENNEWICK	981	1	137
		2	136
		3	153
		9	196
		18	166
		19	169
		34	211
		35	215
		36	214
LULING	1222	1	152
		3	203
		4	187
		5	241
		6	243
JAMES RIVER	630	1	242
		4	233
		8	221
		13	220
		14	200
		19	220
		23	229
		26	222
DAME POINT	1300	1	128
		9	190
		18	170
		19	162
		27	215
		36	232

TABLE 2.6 (Cont'd)

BRIDGE	MAIN SPAN ft.	CABLE NO. REF.	RELATIVE DAMAGE. [UNITY = DAMAGE BY ONE HS 20 TRUCK]
CHAO PHYA	1476	1 8 17 18 26 34	260 264 208 221 252 241
ANNACIS	1525	1 6 12 18 24 25 30 36 42 48	215 168 192 202 189 179 194 189 216 185

2.7 AERODYNAMIC EFFECTS

It is well known that bluff objects placed in fluid flow are subject to a wide range of forces, both in the line of the flow and across it. Some of these forces are quasi static, and some time- dependent, exhibiting characteristics of either regular periodicity or randomness. In the former case, if the frequency of the force is close to a natural frequency of oscillation of the object resonance may occur and the object may be forced to oscillate with a large amplitude. In the case of random excitation, if a significant proportion of the energy of excitation has a frequency close to the frequency of the object, the latter may exhibit oscillations of significant magnitude. The implications of this for stayed girder bridges in wind flow will now be discussed, and some tentative recommendations for dealing with the problems made.

It is necessary to distinguish between the effects of wind excitation on the bridge superstructure as a whole, and on the stays themselves.

2.7.1 Effects on the Bridge Superstructure

2.7.1.1 Quasi Static Effects

The shape of the bridge superstructure determines its drag, lift and pitching moment coefficients. A steady wind will then deflect the superstructure by an amount which can be estimated from the forces generated by these effects. We can find no evidence in the literature that these quasi-static effects occur sufficiently often to have any significant effect on the fatigue of any elements of a structure, including the cables. This is because the elements have to be designed for the maximum static forces resulting from such effects during a structure's lifetime and which therefore occur only very rarely; frequently occurring forces are of an order of magnitude lower than the maxima and even these occur for a number of cycles of loading well below that which is likely to be of significance for fatigue. Such quasi-static effects will thus not be discussed further.

2.7.1.2 Periodic Forces

Periodic forces on the bridge structure can arise from a number of causes:

(a) The alternate shedding of vortices from the upper and lower surfaces of the superstructure gives rise to a regular periodic across-flow force on the structure. It can also give a periodic along-flow force, although this is normally of very small magnitude and hence of no significance.

(b) Randomly fluctuating forces resulting from the fact that the wind is not steady in either velocity or inclination.

(c) Galloping.

(d) Classical flutter.

(e) Stall flutter.

The last three of these are very serious instabilities and, if they should occur on a cable-stayed bridge would be likely to cause general structural failure before any questions of cable fatigue arose. Designers, therefore, are well aware of them and should proportion their structures to avoid any proneness thereto. Such proportioning is a broad subject outside the scope of cable fatigue research, but general guidelines are given elsewhere (Ref. 9).

Whilst careful design of the bridge superstructure can minimise the effects of (a) and (b) above, it is frequently impossible to eliminate them. They can give rise to two effects which are directly relevant to cable fatigue.

The first can arise if the natural frequency of the stay cable is close to that of the structure; if the periodic forces set the structure in motion there is a risk that resonance of the stays will occur, causing large lateral oscillations. This causes large angular changes at the stay anchorages and hence substantial fatigue damage if not stopped. Fortunately it is normally possible to control large lateral oscillations of stays by damping (this is discussed more fully in 2.7.2).

The second effect is that of a pulsating tension in the stays due to bridge vertical or torsional movement. In discussing this it is necessary to distinguish further between the vortex shedding response and the turbulence response, since the latter is only likely to be of significance at high and rarely occurring wind speed. Thus it is unlikely to cause premature cable fatigue and indeed we have found no evidence in the literature that it ever has. Vortex shedding response is different in that with most bridge proportions it would occur, if at all, at frequently occurring wind speeds. In some bridges it may be as low as 10 mph, and for bridges of moderate span is likely to be below 50 mph.

Efforts have been made in Clause 7.1 of the British Draft Bridge Aerodynamic Rules (Ref. 9) to quantify the fatigue damage arising from this cause, and this may then be added to the damage from all other loadings. Considerable further work and indeed research, would however be necessary to put this in a suitable format for inclusion in rules applicable for use in the US. This is because the US rules at present are not framed in terms of calculating explicitly the total cumulative fatigue damage.

However if a designer wished to satisfy himself that he has left an adequate margin of safety to cater for this effect it would be possible for him to refer to the approach outlined in the British Draft Bridge Aerodynamic Rules. He would first of all need to subtract his calculated live load stress range fom the allowable given in this report. He could then convert this stress range into an effective unused fatigue damage which could be expended on the predicted lifetime aerodynamic damage effects. This is not an elegant approach for what may be a very small effect, which is why we recommend further work and research.

2.7.2. Effects on the stay-cables themselves

As with the effects on the bridge structure, the quasi static effects of wind on the stays appear to be of little significance, causing only angular changes at the anchors which are likely to be smaller and much less frequent than changes from other loadings.

There are, however, numerous recorded occurrences of stay oscillations resulting from the aerodynamic excitation of the stays themselves. Unfortunately it is very difficult to detect any consistent pattern in what makes stays prone to oscillation. A few guidelines seem to appear:

(i) Smooth circular section cables (e.g. sheathed or locked coil) are more prone to vortex-excited oscillation than are spiral open cables.

(ii) Spiral open cables are more prone to galloping than are smooth ones.

(iii) Certain cross sectional shapes of cables made up from several strands are more prone to oscillation than others (see (e) below).

(iv) Increased internal damping resulting from certain forms of construction (e.g. short lay length) may inhibit oscillations (but could give rise to other undesirable effects such as fretting).

(v) Icing of stays can change their shape and make otherwise stable stays prone to galloping. At least one stayed mast is known to have collapsed from this cause, although we have no evidence of adverse effects on bridges.

Whilst these effects are difficult, if not impossible, to quantify it is appreciated that excessive oscillation of the stays for a significant period of time will almost certainly lead to premature fatigue damage. Fortunately it has been found that comparatively simple damping systems can reduce or eliminate the effects. A few typical unpublished cases are described below:

(a) Stay cables designed as six separate spiral open strands, interconnected by rubber lined clamps at the third points which could not, of course, be fitted until all the strands had been erected. Until this stage the strands were free to move independently and significant, probably vortex-excited vibrations occurred. After the clamps were fitted the vibrations ceased as a result of interaction of the stays and dissipation of energy in the rubber. Indeed, even before completion, the vibrations had been much reduced by a simple temporary rope tie between the strands.

(b) A number of cases with various cable shapes and configurations where oscillations (sometimes in high modes) have occurred have been stabilised by fitting simple automobile type (dashpot) dampers between the stay cable and bridge deck at road level.

(c) A case has been informally reported of a multi-stayed bridge with sheathed cables which vibrated and where it is proposed to interconnect the stays with thin wires.

(d) There are various reports of guys and suspension bridge hangers which have been stabilised by attaching a Stockbridge damper (essentially a small tuned inertial damper) at an antinode of oscillation.

(e) A bridge with stay cables comprising four strands arranged in a square cross section exhibited a galloping oscillation of such amplitude (3 feet) that the deck was deflected as well. Fitting of dampers as in (b) above stopped this.

2.8 LOCAL EFFECTS ON STAY CABLES

The data collected together with the researchers' own experience on cable-stayed bridges have highlighted many points of interest that were considered during the preparation of this report. These include the following items:-

2.8.1 Sockets

Spiral bridge strand and locked coil cables generally use hot poured white metal or zinc as the socketting material. White metal is lead based and has shown poor creep properties with cable draw occurring. This material is not to be recommended. Zinc, however, has much better creep performance. Spiral bridge strands and locked coil cables are usually laid up with a grease between the wires to provide lubrication and corrosion protection. When the molten zinc is poured into the sockets this grease tends to be driven back up the cable leaving an unprotected zone at the mouth of the socket. By careful heating of the cable it should be possible in some cases to encourage the grease to run back into this sensitive area. We are not aware of this idea being used on any bridge as yet but some preliminary tests have been carried out.

The temperature at which the zinc is poured into the socket has been considered in some cases to affect the fatigue performance of the stay cable. There is very little data in the literature concerning this effect but the most detailed information is given in a paper by K. Kondo et al (Ref. 10) on the design and construction of Toyosato-Ohhashi Bridge. Here, there are 9 small scale fatigue tests plotted (with some additional run out results). Two S-N curves have been drawn through these points. The curves are shown as straight lines on a log-linear plot and they appear to cross over each other at about 80,000 cycles. This would seem to be an unrealistic representation of likely behaviour and the authors themselves accept that it is difficult to predict results for full sized cables based on these small scale tests. It is clear that more research is needed in this area.

Parallel wires and strands are often anchored in their sockets with a filled epoxy resin material. In general these appear to have performed well during their relatively limited period of service. However recent research noted by Tilly (Ref. 11) has shown that fine micro cracking of the epoxy material can occur and lead to corrosion at the resin-to-steel interface.

2.8.2 Corrosion

Modern research has indicated that atmospheric pollution is the prime cause of steel corrosion. Humidity is an essential factor, but on its own it has virtually no effect. It is the presence of pollution, and in particular the presence of chlorides and sulphides, that accelerates the oxidising process dramatically.

With the onset of corrosion the fatigue resistance of the member is greatly reduced. Designs should be detailed in such a way as to reduce the possibility of corrosion starting and to permit early detection if it does occur. In this respect, any system that uses sheathing and grouting is at a disadvantage. Visual inspection will not be able to detect any distress until it has become very severe. There is however sophisticated equipment on the market and in use in the mining industry which claims to be able to detect corrosion beneath the surface of a stay cable system. (Ref. 12) It is believed to be possible to adapt this system, which uses the Hall effect, for application on a larger diameter of stay cable. It cannot, however, be used in the region of sockets. For this area, other methods need to be developed, the most promising of which seem to be accoustic pulsing and possibly accoustic emission.

It should be noted that the survey referred to in section 2.1 revealed several sheathed and grouted stay cable systems that appeared to be suffering deterioration. This survey highlighted the great importance of easy maintenance. If ability and access for routine inspection is marred in any way, the system is bound to have a greater risk of severe corrosion occurring before any remedial action takes place.

Galvanizing is one method of providing effective corrosion protection. It can be applied to the wires either before or after the drawing process. The heating involved can reduce the ultimate strength of the wire by a small amount. However, this small penalty seems very worthwhile when the greatly enhanced corrosion protection is considered. No adverse fatigue effect from galvanising has been recorded in the data collected. Tilly (Ref. 11) reports that although some authorities fear hydrogen embrittlement may be caused by the galvanising process, no such effect has been recorded in the UK where galvanising wires is common practice. Neither has any case in the USA been documented in the literature. It therefore seems that some additional research into galvanizing methods is perhaps necessary to allay any fears that may exist in this direction.

Another corrosion prevention method currently finding favour is epoxy coating of wires and strands. It must be noted that, in some epoxy coatings on flexible members, fine micro cracking of the epoxy material can occur and lead to corrosion of the wire. There is a great risk that this corrosion can be much more severe than that which would occur in uncoated wires because one of the early stages for the epoxy coating process is a pickling procedure which removes some of the outer hard-drawn skin from the wires. This layer includes the burnished grease etc. deposited in the drawing operation, and can provide a certain amount of resistance to the onset of corrosion.

As a conclusion to this brief section on corrosion, it must be noted that guaranteed corrosion protection for stay cables is paramount. There is evidence from in-service performance of many different systems and the vast majority have not performed totally satisfactorily in the provision of fully effective corrosion protection.

Efforts to correct this position have been made and need to continue to be made to allow the designer to specify in confidence that full corrosion prevention can be provided. It is not feasible to try to design stay cables while at the same time allowing corrosion to occur, since quantifying the degree of corrosion becomes an impossible task.

Associated with the prevention of corrosion is the need to maintain and monitor performance to ensure that corrosion is not occurring. This highlights the need to be able to inspect, first of all visually, then through the use of suitable non-destructive testing (NDT) methods. If the visual inspection option is to be prevented (as in sheathed stay cables), then it would seem necessary that the designer and owner should be extra confident on the ability of the system to perform and that a suitable NDT method should exist as back-up. In addition to this, there needs to be a fall-back position if, despite all best endeavours, corrosion does occur.

Here it seems wise and necessary that any new design using cable-stays should provide for easy replacement of any cable-stay without disruption to traffic flow or detriment to neighbouring cables.

2.8.3 Bending effects

Work by Hobbs (Ref. 13) with a commentary by Flint & Neill (Ref. 14) produces some tentative proposals for estimating the effective inertia of spiral strand acting in bending. Hobbs (15) produces an S-N curve for stress ranges due to transverse loading that lies above that for axial stress but he notes that this must be treated with caution. If however, this proves to be correct it would be quite reasonable to take the live load stress range due to transverse loading and add it to the axial load range as recommended by the PTI (Ref 2). This whole subject is, however, extremely complex and requires more research and test results before any firm conclusions can be drawn.

The literature also gives suggested formulae for calculation of bending stresses in parallel-wire and parallel-strand cables but does not produce test results that can be compared with those from Hobbs discussed above.

It should be noted that stresses due to transverse loading can be derived from many sources which include:-

(i) Variation in axial load creating a change in the sag of the cables.

(ii) Direct transverse loading, such as wind loading.

(iii) Variation in the orientation of anchorages (eg. displacements of the deck and hence lower anchorages due to traffic loading. Also rotation of towers due to unsymetrical loading).

In most cases sockets do not rotate (even if pinned or spherical seatings are provided, care has to be taken that the friction developed can often prevent rotation).

Due to the complexity of calculations and reliability in interpreting the results, we recommend that where possible bending effects should be minimised at the design stage. For parallel-wire and parallel-strand cables, where the bending effect is more severe than that in spiral bridge strand or locked coil cables, we note that many anchorage designs incorporate a steel tube at the mouth of the socket. This helps increase the radius of curvature of the cable which thus reduces bending action. Acting in the same way, a trumpet detail can be added to the mouth of sockets for spiral bridge strand or locked coil cables. This was done successfully on the suspenders of the Forth Road and Second Bosporus suspension bridges. (See Fig. 18)

2.8.4 Local biaxial or triaxial stress conditions at clamps and saddles

Saddles and clamps have been used successfully on several bridges and there are no reports of detrimental effects in the literature.

There is a point of detail that should always be observed, however. No sharp corners of castings or fabrications should be allowed to remain in the vicinity of the stay cables. The ends and sides of all saddle troughs and cable clamps should be given as large a radius as feasible to avoid any possibility of a kink effect which would become a prime site for crack initiation. Where space is limited the amount of fairlead provided should be determined by the maximum angle of rotation of the cable in service together with a realistic estimate for the likely setting out errors that may occur.

2.9 DERIVATION OF S-N CURVE FROM TEST DATA

Many fatigue tests on stay cables and their constituent elements have been carried out by various authors (Refs. 13, 19, 20, 21, 22). A large number of test results have been collected together and plotted in Fig 5 providing the basic raw data for this report. These results do not have a consistent failure criterion, they also contain results for single wires and cables of all sizes. This goes a long way to explaining the wide scatter shown on Fig. 5. Results for large cables (cables greater than 30 mm in diameter) are plotted in Fig 6. Stress range divided by ultimate stress is shown on the vertical axis.

It is reported by several authors, including Hobbs (Ref. 13), that the mean stress does not seem to have a significant effect on fatigue behaviour. The reported dominant effect is stress range. This behaviour appears to be borne out in the data collected for this report. Figs 7 and 8 illustrate the point. The data is plotted with highlights on those points where mean stress is less than 25% of ultimate and less than 30% of ultimate, respectively. There is no obvious trend between these two Figures.

Fig. 9 shows a total of 53 results for only the larger scale tests (for cables larger than 30mm in diameter). There appears to be no consistent pattern that distinguishes the results derived from the various cable types. It is interesting to note that the PTI recommendations (Ref. 2) have separated three cable types but that when the stress range is divided by the relevant ultimate stress the recommendations for bars are the same as those for parallel strand.

The 53 test results were analysed and it was noted that in many cases the failure criterion was not consistent. In these cases it was necessary to adjust the life to conform with the chosen failure criterion (rupture of 5% of metallic cross section). This adjustment was made by extrapolation in several ways, depending on the nature of the published results. The adjustment methods included:-

(i) interpolation between two recorded results for the same cable

(ii) interpolation from a single result using other results on other cables.

When all 53 test results on large cables were presented, based on the same failure criterion, it was possible to perform a regression analysis which yielded the following formula for the mean line through the points:-

$$\underline{\text{Log } N + 3.763 \text{ Log } S = 2.989}$$

The slope of this S-N Curve (3.76) has been used in all calculations in this report when the relationship between stress range and fatigue life is required.

A summary of the data and the regression analysis is given in Figs 9, 9A and 9B.

2.10 CUMULATIVE DAMAGE ASSESSMENT

There has been research into the accuracy of Miner's Rule at the Welding Institute (Refs. 15, 16, 17, 18). It is understood that work has also been carried out at the University of Texas. The Welding Institute has observed in some weldments that it is possible to produce differing damage effects from a given set of stress ranges depending on the sequence in which the structure is submitted to the stress ranges. A large stress range can initiate a crack which then propagates under subsequent smaller stress ranges which on their own would have created very little damage.

Miner's Rule does not take account of sequence of stress cycles or mean stress and the research has indicated that it may not be safe in some instances. It is too early at this stage to determine whether the research noted above will yield results that could be applied to stay cables. For weldments simple alternatives to Miner's Rule are being suggested using an analysis method that involves dividing the stress history into blocks of stress patterns. However, because cable stress patterns are of a more random nature, this division would not be an easy operation and we recommend that Miner's Rule should continue to be used for the present.

The computer program specially developed for this project uses a Miner's Rule assumption in its calculation of relative damage effects.

2.11 SPECIFICATIONS

A Draft Guide Specification for the design of cables in cable-stayed bridges has been produced and is presented in Appendix E. A commentary on the materials and manufacturing specification is given in Section 3.2 below.

Many important issues have been raised in earlier sections. These include the recommendation that cable replacement should be possible, and the recommendation that cable corrosion and aerodynamic effects should, as far as possible, be designed out of any structure. In addition, the literature has shown that there have been problems with splitting sheathing, and there may be problems with epoxy cracking in some anchorage system designs.

Specification for testing has been included in the Draft Guide Specification. It may eventually be possible to relate small scale tests to the final behaviour of large stay cables but the data for the test results plotted in Figs 5 and 6 do not seem to indicate that this is likely. As the data to plot Fig. 6 was assembled it was clear that the small scale test results had an S-N curve that was very much flatter than that for larger cables. This will make correlation difficult.

CHAPTER 3

INTERPRETATION, APPRAISAL, APPLICATIONS

3.1 PREPARATION OF CABLE FATIGUE DESIGN PROVISIONS

3.1.1 Introduction

It is generally accepted that the present AASHTO design method for calculating the allowable stress range in members and fasteners subject to repeated variations or reversals of stress in steel structures uses:

(i) An artificially heavy fatigue vehicle (the HS20 vehicle weighs 72 kips - the mean truck weight in the FHWA survey data is 43 kips) together with

(ii) An artificially low number of passages of this vehicle across the structure to be designed. (The designer applies 2 million cycles over the design life of the structure - for a design life of say 80 years this is only 68 loading cycles per day.)

The net damage effect produced by this method of analysis should be compared with real vehicle weight spectra and a realistic number of cycles of loading that would occur during the expected design life of the structure.

At present it seems that the damage effect implicitly incorporated into the AASHTO design method can be more severe than the damage effect that is created by the vehicle weight spectrum taken in this report (see section 2.5) except when saturated traffic flow conditions are reached. In other words, there can be a safety factor on life between the design method and the present best representation of real life.

It should be noted that this safety factor on life will be gradually eroded as the vehicle weight spectrum develops in time, with a bias towards heavier vehicles. (In fact if the mean vehicle weight were increased by about 10% then the AASHTO margin of safety would, in most cases, disappear. It will thus be necessary to review the AASHTO fatigue design method at some stage in the future. For the purposes of this report we have maintained the present nominal AASHTO margin of safety and have remained with the same design method so that figures for allowable stress ranges in cable-stays of cable-stayed bridges can be determined from the present AASHTO format. If, and when, the approach adopted by AASHTO is reviewed, then the approach for cable-stays can be revised to follow suit.

The following chapters show how the recommended allowable stress ranges were developed from the available experimental data, considering first of all the effect on simple short span structures. These results are then adapted through the use of suitable factors, to be then applicable to longer spans and to various widths of carriageway carrying various levels of Average Daily Truck Traffic (ADTT).

3.1.2 Stress-Endurance relationship developed from experimental data

The summary of experimental results shows that the relationship between stress range and number of cycles of loading that can be supported in cable-stays is of the form:-

$$\text{Log } N + K \log S = \text{Constant}$$

where N is the number of cycles to failure

and S is the ratio of stress range to ultimate stress

and the value of K is -3.76 (see section 2.10)

S is directly proportional to the weight of vehicles that create the stress range in the cable-stay being examined.

In addition, the inverse of N is a direct measure of the fatigue damage created by a given load.

In other words, if (for example) the weight of a given vehicle (and hence the stress range) is doubled, then the fatigue damage created is increased by a factor of 13.5 (i.e. $2^{3.76}$).

In order to permit comparison between the AASHTO design method and the effects of our representation of real loading, we have chosen a consistent unit of truck damage. We have used the unit truck damage (UTD) caused by a single HS 20 truck (72 kips), making one single crossing. Note that this is purely for comparison purposes. The size of unit chosen will have no effect on the values of relative damage derived in this work. In other words the unit truck damage caused by a single 72 kip truck is taken as 1, and the relative damage caused by a single 144 kip truck (or two HS 20 trucks side by side) is $(144/72)^{3.76}$. - That is, 13.5. (The effect of doubling the weight of the vehicle increases the relative damage by a factor of 13.5 - as before).

3.1.3 Effect of Traffic on Short Spans (up to 400 ft. long)

The Standard Specification for Highway Bridges (Ref. 3) is intended for design of shorter span bridges up to main spans of

500ft or so. (Note that cable-stayed bridge main spans are typically longer than this - in the range 400ft to 1500ft.) It is therefore assumed in the present Standard Specification that no more than one truck per lane is carried at any one time by these short bridges.

With this assumption it is possible to compute (for short spans only) the damage caused by the train of trucks that has been adopted by this report (see section 2.5).

The calculation is made for the truck train of 372 trucks crossing individually as follows:-

Mean truck weight kips	Number of trucks in train	Relative damage caused by trucks crossing individually Units = HS 20 truck damage
5	18	.001
15	43	.12
25	79	1.48
35	52	3.45
45	36	6.15
55	36	13.1
65	48	32.7
75	38	44.3
85	16	29.9
95	4	11.3
105	1	4.1
115	1	5.8
	372	152.4
	===	=====

Hence the damage on relatively short bridges (with main spans of no more than 400ft) caused by the train of 372 trucks crossing individually is 152.4 times the unit truck damage caused by a single HS 20 truck.

A comparison can now be made between the fatigue damage effect intrinsically incorporated into the present AASHTO design method, and that which will occur in real life on various widths of bridge carrying various intensities of average daily truck traffic (ADTT). This will allow an estimate of the margin of safety on life that is implicitly used when the present AASHTO design method is followed.

For the purposes of this comparison a cable-stayed bridge is considered with two planes of stays. The assumption is made that the fatigue damage effect felt by one plane of stays is that contributed by the loading on the carriageway (the half bridge width) adjacent to the stays in question. (In reality there will be transverse distribution of live loading between the two planes of stays, with the result that the effect from the adjacent carriageway will be reduced to some extent, but there will be a complimentary additive effect contributed by the traffic on the second carriageway. In general these two effects can be considered to be of about the same magnitude and the assumption is therefore reasonable.)

The AASHTO method requires HS20 trucks to be placed in a specified number of lanes to produce maximum stresses. If the bridge carries a total of 4 or more traffic lanes irrespective of direction then 75% of the lanes are required to be filled each simultaneously with a single HS 20 truck in order to derive the maximum stress range induced (clause 3.12 of the AASHTO Specification).

Following the same logic as that adopted for the trains of trucks, we have assumed that one plane of stays carries one carriageway of HS 20 loading. It then follows that, in order to derive the maximum effect, each plane of stays would be designed to carry a single carriageway with an HS 20 truck placed simultaneously in each of its traffic lanes to give maximum stresses.

The AASHTO design method identifies a critical value of ADTT in one carriageway. When the density of traffic reaches 2500 trucks per day in one direction then the number of cycles of truck loading to be carried by longitudinal members is increased from 500,000 to 2,000,000. The relative fatigue damage that occurs at this transition position has here been derived and so the implicit safety factor incorporated into the AASHTO design method has been deduced. A single pass of a single HS 20 truck has been used as the unit truck damage (UTD) as noted earlier. Hence the following table of relative damage for short spans (up to 400ft) can be calculated:-

TABLE 3.1.3 RELATIVE DAMAGE FOR SHORT SPANS

Number of lanes on bridge A	Assumed Number of HS20 trucks carried on one plane of stays B	Relative damage for 2 million cycles of max. HS20 loading (millions) AASHTO DAMAGE C	*Relative damage for trains of trucks giving ADTT of 2500 for 80 years (millions) REAL DAMAGE D	Implicit AASHTO safety factor on life E
4	2	27.1	24.6	1.10**
6	3	124.5	-	-
8	4	367	-	-

Column B is 50% of Column A. Column C is $(\text{Column B})^{3.76}$ x 2 million

*The calculation of the relative damage effect for trains of trucks giving an ADTT of 2500 has been made with the following assumptions:-

(i) Relative damage for 1 train of 372 trucks is 152.4 (as calculated earlier in this section).

(ii) Design life taken as 80 years, composed of 50 effective weeks each of 6 effective days of full ADTT. Hence number of trains of trucks to be carried during design life with ADTT of 2500 is 2500 x 6 x 50 x 80/372 = 0.161 million trains of trucks. Hence relative damage is

0.161 x 152.4 = 24.6 million

(iii) The damage effect is not entered for 6-lane and 8-lane highways because ADTT of 2500 per carriageway is often an unrealistically low truck density for these sizes of carriageway.

**Note that the factor of 1.10 on life is derived by dividing column C by column D and is equivalent to a factor of 1.03 on stress range.

It can be seen from Figure 12 (derived later in section 3.1.7) that, over the range of traffic flows on two-lane carriageways, the present AASHTO design method over-estimates the damage for some flows and under-estimates the damage for the remainder of flows. The factor of 1.1 on life derived above is the largest safety factor between the AASHTO method and our representation of real damage and has been adopted as the safety factor on life for the remainder of this report.

3.1.4 Effect of Traffic on Longer Spans (400 ft. - 1500 ft.)

As the next stage in development of the calculation we now consider the longer spans (400ft - 1500ft) that are more common for cable-stayed bridges. The calculations for short spans made in the previous section assumed that there was never to be more than one truck of the truck train on the bridge at any one time. For the longer spans now to be considered this assumption becomes unrealistic. It is possible for trucks to cross longer spans in bunches so increasing the damage caused.

As described in section 2.6, the effect caused by truck trains has been analysed using computer modelling. The design truck train (giving mean damage from 20 randomly generated truck trains) was run across various structural configurations of typical cable-stayed bridges. This showed that the majority of relative fatigue damage is caused by bunches of trucks within the train. Typically the relative fatigue damage effect caused by one train of trucks is about 200 times that caused by a single HS 20 truck. The actual figure varies from structure to structure and is also influenced by the location of the particular cable in question. The range of values, determined from the structural models created, was from 130 to 270 (see Table 2.6). In other words, the truck bunching effect typically increased the damage effect by a factor equal to 200/152.4 = 1.31.

But this bunching factor can vary from

130/152.4 = 0.85.

to

270/152.4 = 1.77.

This range of values encompasses many effects including the effect of the location of the stay within the system of stays, any effect that the form of bridge may have and any span effect that may exist. The magnitude of these various effects was not well defined, making it difficult to consider them separately. As the variation in the size of this bunching factor is small, it is reasonable to group the various effects together.

It should be noted here that it was expected that there would be a span effect. The reason that it is found not to be significant for spans up to 1500ft. is because the majority of damage is caused by bunches of trucks and these bunches are so spaced that no more than one bunch of trucks lies on any influence line at any one time.

3.1.5 Total Effect on Dual Two Lane Highways

It can be seen, from sections 3.1.3 and 3.1.4 above, that for dual two lane highways, and smaller roads, the philosophy incorporated into the present AASHTO design method can be preserved if the real damage effect is factored twice:-

(i) by 1.10 - to represent the implicit AASHTO safety factor for an ADTT of 2500 - and

(ii) by a figure between 0.85 and 1.77 - to represent the effect of bunching trucks on longer spans.

The combined effect of these two factors varies between:-

0.94 - 1.95 on life

or

0.98 - 1.19 on stress range.

Considering the accuracy of the available data and the scatter in results, it seems difficult as noted in section 3.1.4 to justify any partition of the factor range (0.98 - 1.19) into zones corresponding to the various bridge types and / or the location of the cable within the cable-stay system. With a total variation of only 20% on the factor, any such partition would tend to be obscured by individual scatter bands. We therefore recommend, at this stage, that there should be no distinction between the various cable-stayed bridge types or between the effect that the location of the cable has within the cable-stay system. Future research and test results may make it possible to make this distinction with more confidence. As an example, however, it is clear that the back-stay cables tend to attract a slightly greater relative damage when compared with internal cable-stays.

Thus, to summarise, starting with the design S-N curve of test results (mean minus two standard deviations), it is possible to derive the design values given in the table below for Dual two-lane highways carrying an ADTT of 2500 per carriageway: -

Cycles of max. HS 20 truck loading (millions)	Stress Range/Ult.Stress		Suggested Design stress range using ultimate stress of			
	Test results design S-N curve (mean - 2SD)	Suggested design Values for use with present AASHTO design method	270-250 ksi	250-235 ksi	220-200 ksi	150 ksi
			7 wire strand Un-coated A416	Un-coated wire A421	Bridge strand Gal-vanised A586	Steel bar Un-coated A722
0.1	.2062	.21-.17	57-43	52-41	46-35	31-26
0.5	.1345	.14-.11	37-28	34-27	30-23	21-17
2.0	.0930	.09-.08	26-20	24-18	21-16	14-12

TABLE 3.1.5 DESIGN STRESS RANGES FOR DUAL 2 LANE CARRIAGES AT ADDT OF 2500 PER CARRIAGEWAY

Note that this table is only relevant for dual two-lane carriageways carrying an ADTT of 2500 per carriageway. The following sections will develop the calculations to include a full range of ADTT levels on wider highways.

It should also be noted that, in line with current research findings, no fatigue limit has been incorporated in the above table. Until further studies in this area have yielded more reliable results it is considered unwise to incorporate any such effect, if indeed any exists.

3.1.6 Effect on Highways Wider than Dual Two Lane Carriageways

As noted above, the damage effect caused by a more realistic density of traffic, especially on wider highways, must be taken into account. For this analysis several densities up to saturation truck traffic have been considered.

The parameters for saturation truck traffic have been taken as follows:-

(i) Limiting total traffic volume assumed to be 20,000 vehicles per lane per day.

(ii) For heavy traffic flow, truck density is taken as 10% of vehicle flow.

From this, saturation truck traffic is calculated as follows:-

Number of lanes in one carriageway	Saturation truck traffic (Average daily in one carriageway)
2	3600
3	4800
4	6400

TABLE 3.1.6A SATURATION TRUCK TRAFFIC

Earlier work (Section 3.1.3) on dual two-lane carriageways considered an ADTT of 2500. When saturation traffic flow of 3600 is reached it is clear that the present AASHTO design method will give an underestimation of the real damage that can occur.

It is therefore necessary to consider varying levels of ADTT on various widths of highway.

The approach adopted in Section 3.1.3 will be followed in this analysis. In other words, an initial damage calculation can be made with the ADTT flow considered to cross the structure one truck at a time. The damage effect that is then calculated can afterwards be modified to create a realistic safe representation of real traffic damage effects. The modification can be achieved by applying three factors to the simple initial damage calculation:-

Factor 1 is the safety factor implicity included within the present AASHTO design method. Our estimate of this factor is 1.10 on life - see Section 3.1.3 above.

Factor 2 is the factor on damage to allow for the grouping of trucks together into bunches on longer spans with the result that more than one truck can be carried on a structure at any one time. This bunching factor has a range of values between 0.85 and 1.77 on life - see Section 3.1.4 above.

Factor 3 is a further factor required to model the additional effect of heavy traffic flow on wider highways. The reasons for, and development of, this third factor are now covered in more detail:-

3.1.6.1 Development of a third factor to model the additional effect of heavy traffic on highways with 5 lanes or more.

The analysis up to present has concentrated on dual two-lane carriageways. As saturation traffic flow is reached on a two-lane carriageway one can consider the effect that the same amount of traffic flow would have if it was travelling on a wider highway with more traffic lanes.

It can be seen intuitively that the damage effect would not vary greatly between the two cases. If there is a difference it is to be expected that, as the wider highway would promote more freely flowing traffic which would be less inclined to form into bunches of trucks, the same level of ADTT carried on a wider highway would result in a slightly lower damage effect. However it is difficult to quantify this effect and we will consider , for the purposes of this work, that a given level of ADTT has the same damage effect, even if it is carried on a wider highway.

Hence for ADTT up to saturation level on two-lane carriageways (3600 trucks per day) the damage effect simulated by the computer modelling of two-lane traffic flow has been used.

However, when ADTT levels in excess of 3600 are considered it is necessary to evaluate the effect on highways with more than two lanes.

The computer model is correct for two lanes of traffic. One can consider the hypothetical case of twice as much traffic on twice as many lanes. In this case it is clear that there will be two extreme conditions to examine, each of which generates its own enhancement factor to be applied to the basic loading model:-

Case (i) If no trucks in the first two lanes coincide with any of the trucks in the two additional lanes and cannot be considered to have added to any bunches outside their own two lanes, then there will be no enhancement factor on the basic single truck analysis.

Case (ii) If each of the trucks in the first two lanes coincides with a truck of similar weight in the two additional lanes, then the weight of each bunch of trucks in the simple two lane analysis will be doubled for this four lane example but the number of cycles will be halved. In this case the enhancement factor on damage caused by this doubling up of traffic will be 6.7 (i.e. $2^{3.76} \div 2$).

The actual value for the third factor clearly lies between the two extreme values of 1.0 and 6.7 described above. A visual representation of this is shown in Fig. 11.

For the purposes of this report (and in the absence of any recorded data on this effect) we suggest that a figure of 75%

coincidences should be considered at this stage. The additional enhancement factors that will be adopted for levels of ADTT in excess of 3600 are therefore as follows: -

ADTT	Additional enhancement factor
4800	2.3
6400	4.6

TABLE 3.1.6B ENHANCEMENT FACTORS ON LIFE

Note that the table of figures that follows in section 3.1.7 shows that 100% coincidences can be considered on wider highways and its effect is still less severe than saturation loading on dual twin carriageways.

3.1.7 Summary of effect on various highway widths, carrying various intensities of truck traffic for cable-stayed bridges using twin planes of stays

The table of figures that follows summarises the relative damage effect for bridges with twin planes of stays carrying carriageways with 2, 3 or 4 lanes of traffic. Various indicative levels of ADTT have been chosen up to the saturation levels relevant to each width of carriageway.

The relative damage effect on short spans has been calculated using the approach as described in section 3.1.3.

i.e. - relative damage for 1 train of 372 trucks is 152.4 UTD

- design life is taken as 80 years, composed of 50 weeks each of 6 days of full ADTT.

This calculated relative damage effect has then been factored by three separate factors on life as described in the previous section:-

Factor 1 (safety) = 1.10

Factor 2 (Bunches) = 0.85 - 1.77

Factor 3 (High ADTT) = 1.0, 2.3 or 4.6

Application of these factors leads to the best estimate for real relative damage effect that is likely to be felt by one plane of stays on a bridge with two planes of stays.

The last column in the following table gives the design damage effect that will result when the present AASHTO design philosophy is followed. Note especially the change in numbers of applied cycles of truck loading from 0.5 million to 2.0 million as the density of truck traffic increases past the ADTT level of 2500.

TABLE 3.1.7A SUMMARY OF DAMAGE EFFECTS FOR BRIDGES WITH TWIN PLANES OF STAYS

Number of lanes in carriage way	Average ADTT over 80 year design life	Damage Effect on Short spans in UTD (cf present AASHTO) Millions	Safety Factor implicit in present AASHTO Factor 1	Factor from bunching trucks Factor 2	Factor for additional bunching in dense traffic Factor 3	Total estimated real Relative Damage Effect	Calculated Damage Effect using present AASHTO philosophy
A	B	C	D	E	F	G	H
2	500	4.92	1.10	0.85-1.77	1.0	4.6-9.6	6.8
	1500	14.75	"	"	"	13.8-28.7	"
	2500	24.58	"	"	"	23.0-47.9	6.8/27.1
	3600	35.40	"	"	"	33.1-68.9	27.1
3	700	6.88	1.10	0.85-1.77	1.0	6.4-13.4	31.1
	2500	24.58	"	"	"	23.0-47.9	31.1/125
	3600	35.40	"	"	"	33.1-68.9	125
	4800	47.19	"	"	2.3	101-211	125
4	1000	9.83	1.10	0.85-1.77	1.0	9.2-19.1	91.8
	2500	24.58	"	"	"	23.0-47.9	91.8/367
	3600	35.40	"	"	"	33.1-68.9	367
	4800	47.19	"	"	2.3	101-211	367
	6400	62.93	"	"	4.6	271-564	367

Column C is computed as follows:-
80 years x 50 weeks x 6 days x ADTT (in column B) x 152.4/372 UTD

Column G is the multiplication of columns C x D x E x F

Column H is computed as follows:-
Number of HS 20 trucks (to give max. loading) raised to the power of 3.76 times the number of cycles of this max. loading (0.5 million or 2.0 million).

When a comparison is made between the damage effects in columns G and H in the above table, it can be seen that, for dual two-lane carriageways, the present AASHTO philosophy produces relative damage effects which are less than the estimate of the real effects (including safety factor) for nearly all levels of ADTT. When dual 3- and 4-lane carriageways are considered, this deficiency is less pronounced but is still present, especially for the higher levels of ADTT.

The comparison between the damage effects in columns G and H is plotted in Figure 12.

The ratio between these two sets of calculated relative damage effects is presented in Figure 13. This shows clearly that the damaging effect on 2-lane carriageways is seriously underestimated by the present AASHTO philosophy and that a factor of up to 7 on life (1.7 on stress range) is required to compensate for this fact. For dual 3-lane carriageways the factor is up to 1.7 on life (1.2 on stress range) and for dual 4-lane carriageways the factor is up to 1.5 on life (1.1 on stress range).

The most extreme underestimate of real damage effects occurs in the low levels of ADTT up to 2500. The short-fall can be relieved, to some extent, if the concession on reduced application of cycles of HS 20 truck loading given in AASHTO table 10.3.2A is changed from the ADTT level of 2500. The level at which 2 million cycles of HS 20 truck loading must be carried can be chosen so that the ratio of real damage to calculated damage is no more than that which exists at higher ADTT levels. Hence an ADTT of 900 has been chosen. This reduces the ratio for dual two lane carriageways to 2.5 on life (1.3 on stress range).

Figure 14 shows the effect that moving the critical ADTT from 2500 to 900 has on the plot of the ratio between real damage effects and calculated damage effects when the present AASHTO design philosophy is followed.

With this small revision to the AASHTO design philosophy, it can be seen that if a total factor on damage of 2.54 is applied to life(1.28 on stress range) then the AASHTO method can be applied and will yield safe calculated design damages for cable stays of twin plane cable-stayed bridges.

The factor of 2.54 is the upper end of the range of factors which has as its lower bound the factor of 0.76 on life (0.93 on stress range).

Hence a table of allowable design stress ranges, based on the present AASHTO design method, has been drawn up in a similar way to that produced in section 3.1.5 but which will cover various levels of ADTT on various widths of highway for twin plane cable-stayed bridges. This table is presented below: -

It should be noted that stress levels for steel bars have not been indicated since no steel bar endurance tests were included in the S-N data presented in Fig. 9. If at some stage in the future more endurance data for steel bars becomes available then it may be possible to confidently provide some guidance for steel bars. It should be remembered that bending effects on bars may be very significant.

Cycles Millions	Stress Range/Ult. Stress		Suggested design stress range using ultimate stress of		
	Cable Test Results. Design S-N Curve. (Mean-2SD)	Suggested Design Values for use with present AASHTO design method	270-250 ksi	250-235 ksi	220-200 ksi
			7 wire strand Uncoated A416	Uncoated wire A421	Bridge strand Gal-vanized A586
0.1	.2062	.22-.16	60-40	55-38	49-32
0.5	.1345	.14-.10	39-26	36-25	32-21
2.0	.0930	.10-.07	27-18	25-17	22-15

TABLE 3.1.7B DESIGN STRESS RANGES IN CABLES

OR TWO PLANE CABLE-STAYED BRIDGES

Note that this table gives the design stress ranges in the cable-stays. This includes axial stress and any allowance for bending stress, as suggested in section 2.8.

If the above table of stress ranges is compared with those permitted for the different classes of detail in AASHTO, (see Figs. 15, 16, and 17) it is clear that the following classifications could be recommended for 2 million cycles on redundant twin plane cable-stayed bridges: -

7 wire strand, uncoated	- Class B	)	For
(ASTM A416)		)	
		)	2 million
Uncoated wire	- Class B	)	
(ASTM A421)		)	cycles on
		)	
Galvanized Bridge Strand	- Class C	)	twin plane
(ASTM A586)		)	
		)	cable-stayed

However it should be noted that the slope of the S-N curves used in the AASHTO classifications is not the same as the slope obtained from the summary of experimental results on cable-stays. For this reason the above classifications are only strictly true for the application of 2 million cycles of truck loading. It can be seen that they may need modification in some cases if fewer numbers of cycles are applied. It is likely that the majority of cable-stayed bridges to be built in the United States will be on Freeways, Expressways or major Highways and Streets, these bridges will then fall into case I or case II of the AASHTO table 10.3.2A entitled "Stress Cycles". With this assumption it is thus only necessary to consider two possible numbers of cycles of truck loading (2 million and 0.5 million). For these two cases it can be seen from the S-N curves (Figs. 15, 16, and 17) that it is reasonable to use the above classifications for 2 million cycles, but that for 0.5 million cycles, it is necessary to classify all wire cables as Class C.

If the AASHTO classifications are to be used then we suggest that for redundant structures, all wire cables should be classified as Class C. This would provide some additional margin of safety especially for parallel wire and parallel strand cables which is felt to be desirable to allow for the present degree of uncertainty concerning integrity of grout within sheaths and ability to inspect for corrosion and broken wires within sheaths (discussed in more detail in sections 3.2.1.3 and 3.2.1.4).

However it is recommended that a tighter specification is produced and that new classifications of detail should be created to give cable-stays their own allowable fatigue stress ranges to be inserted into a table similar to table 10.3.1A of the AASHTO specification. This philosophy avoids the anomolies and additional conservatism created when one endeavours to fit test data into the existing AASHTO classifications which have a difficult slope of curve. It also allows easy incorporation of any benefit when additional endurance results become available.

3.1.8 Loading Effect on Cable-Stayed Bridges with a Single plane of cables

All the foregoing work in this chapter has been based on bridges with twin planes of cable-stays. Assumptions were made which effectively meant that the truck loading from one carriageway was carried on one plane of stays.

When considering bridges with a single plane of stays these earlier assumptions need to be carefully reassessed and revised as necessary.

The effects of truck loading from a single carriageway on bridges with twin planes of stays was developed from short spans, to longer spans with bunching of trucks and high levels of ADTT on wider roads. The reasoning is just as valid for bridges with single planes of stays as it is for bridges with two planes of stays.

The differences between the analysis of the two types of bridge occur in two main areas: -

3.1.8.1 <u>The application of the AASHTO design method</u>.

For the theoretical design method, the magnitude of the load carried by the cable is determined by the designer in his application of HS 20 truck loading. The single plane of stays carries the full bridge width of live load which, for bridges with 4 or more lanes is determined by taking 75% of the number of traffic lanes and placing an HS 20 truck in each of these lanes. In order to compare this loading with that taken in earlier sections of this chapter, for twin planes of stays, it should be remembered that it was assumed that a full carriageway of load (50% of the bridge width) was carried on one plane of stays. Hence it can be seen that the single plane of stays will be designed to carry more live load, in the ratio of 75/50 - i.e. 150% of the load carried by a cable in a twin stay system. In other words, the AASHTO design method will automatically increase the loading (but not necesarily the stresses) on a single plane system by a factor of 1.5.

3.1.8.2 The application of real truck loading.

The increase in truck loading experienced by a single plane of stays when compared with a cable in a twin plane system is a function of the number of coincidences of loading travelling in opposite directions. It has already been established that the majority of fatigue damage to cable stays in real life is caused by bunches of trucks.

If one assumes that each carriageway is carrying the same ADTT, then for a single plane of stays, there are two extreme cases of loading to be considered: -

Case 1: if the truck loading in each carriageway is such that, for the cable under consideration, the loading in one carriageway does not add to the loading caused by any of the bunches of trucks in the other carriageway, then the increase in loading can be considered as merely a doubling of the ADTT. In other words the fatigue damage is multipled by a factor of 2.0 (or the stress range is factored by 1.20).

Case 2: if the truck loading is such that, for the cable under consideration, every bunch of trucks in one carriageway coincides with a bunch of a similar weight in the other carriageway, then the loading is factored by 2. Hence the fatigue damage is increased by a factor of 13.5 (i.e. $2^{3.76}$).

These two cases are obviously extreme examples, neither of which may necessarily occur. Case 1 will clearly underestimate the true effect, while Case 2 will greatly overestimate the situation.

Unfortunately, it is believed that there is no available recorded data from existing bridge sites that would confidently permit determination of the actual factor on loading. This is clearly an area that would benefit from future research and would be relatively simple to carry out. In the meantime, it is suggested that a reasonable estimate can be made for the purposes of this report.

The problem of choosing a realistic factor is similar to that addressed in section 3.1.6.1 above for the effect of heavy traffic flow on larger highways. In that case a doubling of traffic lanes running in the same direction was considered. In this case we are considering a doubling of traffic lanes, but with the traffic running in opposite directions. In that case a figure of 75% coincidences was taken. In this case with traffic travelling in opposite directions (with a large relative speed) the chances of coincidences occuring at the peak of any given influence line are obviously reduced. At this stage, we suggest that it is reasonable to use a factor of 1.5 on loading for bridges with a single plane of cables. This factor lies in the range of 1.2 - 2.0 outlined above, although it may slightly over estimate reality.

It can now be seen that both the present theoretical design approach, and the best estimate of real-life truck loading effects produce the same enhancement factor of 1.5 on loading when single plane cable-stayed bridges are compared with twin plane bridges. In other words, the theory models the practice very closely and therefore there is no need to provide any additional correction factor when stays in single-plane systems are being designed. This simplifies the process considerably, meaning that the same set of factors derived earlier in this chapter is equally applicable to both types of cable-stayed bridge.

The recommendations for fatigue design of cables in cable-stayed bridges can now be summarised.

3.1.9 Summary of recommendations for fatigue design of cables in cable-stayed bridges

It was noted above that the slope of the S-N curves for cables in cable-stayed bridges is different from that adopted by the various categories of fatigue detail incorporated in the AASHTO specification.

There are two approaches that can be followed in order to create recommendations for fatigue design of cables in cable-stayed bridges.

Firstly, the best existing AASHTO category can be taken for each of the common types of cable-stay.

Secondly, new categories of fatigue detail could be created for each cable type with their own set of allowable stress ranges for different numbers of cycles of truck loading.

The second approach is the recommendation of this report since the first is bound to be more conservative, especially at the higher number of cycles.

3.1.9.1 Recommendations using new classification of fatigue catagories

The discussions on previous pages have shown that the figures given in table 3.1.7B can be regarded as valid for all types of cable-stayed bridge. We therefore recommend that the lower bound of the design stress ranges given in that table should be generally adopted. It should be noted that there is only 1ksi difference between the performance of parallel wires and parallel strand.. We therefore recommend that these two should be grouped together.

The recommended allowable design stress ranges for cable-stays in cable-stayed bridges can be summarised as follows:-

Type of Cable-Stay	Design Stress Range ksi			
	Redundant		Non-redundant	
	For 500K Cycles	For 2000K Cycles	For 500K Cycles	For 2000K Cycles
7 wire strand, uncoated (ASTM A416) Uncoated wire (ASTM A421)	25	17	17	14
Galvanized Bridge Strand (ASTM A586)	21	15	15	12

TABLE 3.1.9A RECOMMENDED DESIGN STRESS RANGES FOR CABLE-STAYS

Note: The difference between redundant and non-redundant systems has been maintained at the level specified in AASHTO Table 10.3.1A.

Note: For the majority of modern cable-stayed bridges, the stays will normally form part of a redundant system. There will, however, be a need in such systems to assess the dynamic effects when one cable is considered to break. There will also need to be an assessment of the fatigue damage incurred by adjacent cables at the time when redistribution of load occurs, following the assumed breakage. If these effects are considered to be small, then the Redundant column of Stress Ranges can be used. If they are uncertain, it may be that the designer would wish to assume non-redundancy. For this reason, an additional margin of safety has been incorporated by reducing stress ranges in this column by an amount equal to that adopted by the AASHTO Specification in Table 10.3.1A.. Further research might allow confidence to specify tighter categories in the future.

It is recommended that the number of cycles of stress to be considered should be obtained from the table below for the appropriate road type.

Stress Cycles				
Cable Stays in Cable-stayed Bridges				
Type of Road	Case	ADTT*	Truck Loading	Lane Loading
Freeways, Expressways, Major Highways, and Streets	I	900 or more	2,000,000	Not applicable for cable stays
Freeways, Expressways, Major Highways, and Streets	II	less than 900	500,000	
Other Highways and Streets not included in Case I or II	III		500,000	

* Average Daily Truck Traffic (one direction)

TABLE 3.1.9B STRESS CYCLES

3.1.10 Effect of possible future increases in weight of HS 20 design vehicle

It should be noted that all the calculations performed in this chapter have assumed that the designer using the AASHTO Specification takes the HS 20 vehicle as his fatigue vehicle and that this vehicle weighs 72 kips.

It is understood that it is now common practice when designing structures on important trunk routes, to increase the specified static design loading requirement by applying an enhancement factor on the HS 20 truck loading.

If this same enhancement factor is applied to the fatigue loading calculations it will clearly have an effect upon the calculations performed in this chapter. The apparent underestimate of damage calculated by the AASHTO design method will be increased by this enhancement factor raised to the power of 3.76. This effect is summarised in the table below for various enhancement factors: -

Enhancement Factor on HS 20 truck loading	Factor by which calculated real cable-stay fatigue damage is increased
1.25	2.3
1.50	4.6
1.75	8.2
2.00	13.5

TABLE 3.1.10 EFFECT OF INCREASING HS20 TRUCK LOADING

Variations on the recommendations have not been produced to accommodate these possible enhancement factors on the HS 20 truck loading, but it is strongly advised that the designer should ensure that he has adopted a consistent approach, preferably by using unfactored HS 20 truck fatigue loading together with the recommendations on allowable stress range given in section 3.1.9 above.

An alternative way of incorporating these enhancement factors could be to regard the increase in weight of the fatigue vehicle as effectively reducing the fatigue category of the cable-stays. For example if the factor applied to the HS 20 truck fatigue loading is 1.25 then the AASHTO fatigue category is effectively reduced by one category. If the factor is 2.0 then the AASHTO fatigue category is reduced by something more than two catogories. This information is given to illustrate the effect. It is not advocated that it should be adopted. It is considered that the recommendations given in section 3.1.9 using unfactored HS 20 truck loading are the best approach to the fatigue design of cables in cable-stayed bridges. It is these recommendations that are reproduced in the Guide Specification presented in Appendix E.

3.2 SPECIFICATIONS FOR MATERIALS AND TESTING

3.2.1 General Principals behind the Guide Specification (See Appendix E)

3.2.1.1 Design Life

Control of manufacture is of vital importance to the long-term integrity of stay systems for bridges. It is understood that the current AASHTO Specification was drafted on the basis that bridge components must be designed for an 80 year life.

3.2.1.2 Stages of Manufacture

Most stay systems in current use today around the world have features which are particularly sensitive to deviation in manufacturing quality. 'Manufacturing' in this sense is deemed to cover the four fundamental stages which apply to metal and other components in bridge construction, namely:

(i) Material product
(ii) Fabrication
(iii) Protection
(iv) Transport and erection.

In structural steelwork, which is thoroughly covered by Division II of the current AASHTO Specification (Ref. 3) the four stages follow naturally in chronological order. In the case of cable-stay systems the stages are not so clear cut, owing to the variety of stay forms and methods of manufacture and installation in current use. For example a bridge strand or locked coil strand will generally be manufactured and protected fully before it leaves the manufacturing works. The most important part of the protective system, the zinc coating is often applied even before the final drawing of the wire. Any blocking agent, which is a second line of defence against corrosion, is applied during the laying up process.

In contrast to this, a parallel wire or strand stay may be assembled from individual wires or strands on site at the erection stage in its final position. 'Fabrication' in effect being done as part of the erection procedure. Protection, in that case, will usually be by grouting within a sheath as the final operation.

3.2.1.3 Special Features of Cable Stays

Most stay systems in use today are proprietary designs with patented features belonging to the original manufacturer. The controls used during manufacture have evolved over the years by a process of trial and error and are very particular to each design. This makes the drafting of a comprehensive specification a difficult task. If the specification is written in too detailed a form there is a danger that certain systems may either be precluded or inadequately dealt with.

Cable stays have three fundamental features which distinguish them from other structural members in a bridge. Firstly, they consist of very small and very highly stressed components. This means that they tend to be more sensitive to manufacturing defects and damage in service. Secondly they are not rigidly encased in a dense material such as concrete which, as in the case of prestressing cables, provides protection against stress fluctuations (both axial and bending) and corrosion. Thirdly the majority of the critical components cannot be readily inspected in service without dismantling all, or part, of the stay. This applies particularly to anchorage regions and in the case of sheathed stays, the complete system.

3.2.1.4 Importance of Corrosion Protection

Although this report is primarily concerned with fatigue of cable stays, the control of corrosion must be taken into account. This is because the fatigue life is significantly reduced if corrosion pitting of the wires is allowed to occur early in the stay's life. As the proposed design rules in Section 3.1 of the report do not make a specific allowance for corrosion, choice of protection system and controls on workmanship on stays are more critical than in other bridge components. This applies particularly to stays which have evolved from prestressed concrete technology where the original performance requirements were significantly different from those for bridge stays (as mentioned in 3.2.1.3).

3.2.1.5 Stages of Product Control

There are four essential stages of product control for cable stays namely:

a) Procedure testing to establish manufacturing techniques

b) Acceptance testing of a proprietary design

c) Quality control testing of the actual components and assemblies for a particular construction contract

d) Maintenance procedures.

3.2.1.6 Procedure and Acceptance Testing

Procedure and acceptance testing must be fully documented so that the conditions under which the manufacture and testing of prototypes has been carried out are permanently recorded. All testing must be independently witnessed by a certified authority so that documentation can be offered as a basis for acceptance of the same design and manufacturing method for subsequent contracts. The documentation of the design details and manufacturing process actually used and the material properties actually obtained for the acceptance and procedure tests must be very detailed. Otherwise it will not be possible to assess the effects of subtle changes in detail design, physical properties or process which might affect the fatigue performance, either directly through mechanical deficiency or indirectly as a result of breakdown in corrosion protection.

The detailed drawings of the stay system, and the results of the procedure and acceptance tests would form an important part of a Contractor's Method Statement for the manufacture, testing, erection and protection of the stay system in question. If the manufacturer of the stay has an Approved Quality Assurance System then his Quality Manual would form an important part of the Method Statement. It therefore seems very desirable that a Method Statement be submitted as part of the tender documents for every cable-stayed bridge.

3.2.1.7 Production Quality Control Testing

Production quality control testing is a matter of routine inspection and destructive and non-destructive testing on components and processes throughout each contract. Some of these have already been standardised for example by the American Society for Testing and Materials and the relevant Standard need only to be invoked by AASHTO. Some will need to be spelled out in detail. Others may arise from the requirements of a particular design and will be defined in the Quality Manual by the particular Contractor or sub Contractor concerned.

3.2.1.8 Maintenance Procedures

It is also proposed that the Method Statement include a proposed maintainence schedule. This would include the Contractor's proposals as to how the maintenance authority would gain access to the various parts of the stay system, including anchorages, and at what recommended intervals, what methods of inspection should be used and what remedial (retrofitting) protection should be applied to ensure that deterioration of the stay is prevented.

3.2.2 Scope

3.2.2.1 Types of Stay System

It is proposed that all basic types of stay system in current use today should be covered by the AASHTO specification for fatigue design of cable-stays. TABLE 3.2.1 shows the basic systems which have been or are in current use around the world for cable-stayed bridges. Other systems may be acceptable provided they are supported by documentary evidence to justify their safe use.

However the shortage of endurance data for parallel bar systems has not permitted us to specify allowable design stress ranges for stay systems using steel bars. (See para. 3.1.7)

The manufacture of stays fabricated from rigid structural steel members, which are rarely used, and only then in fairly short lengths, are assumed to be adequately covered by the existing provisions in Section 10 of Division II of the AASHTO Specification. (Ref. 3).

3.2.2.2 Components

The prime components of the stay system which need to be covered by new AASHTO provisons can be seen from TABLE 3.2.1. These are the cable, the anchorage and the protection systems. In addition to these there will be ancillary components such as saddles, clamps, dampers etc., which do not participate directly in the load transfer or corrosion protection function of the stay.

TABLE 3.2.1 BASIC CABLE-STAY SYSTEMS

CABLE			ANCHORAGE		PROTECTION	
Element Configu-ration	Element	Shape	Element End Connection	Bonding Agent	Sheath	Filling Agent
Parallel	Wires	Round	Button head	Cement Grout or Epoxy Resin or none	Loose Polyethylene tube* or Steel Pipe	Cement Grout or Grease or Pitch Epoxy or wax
	Strand	Hexagon (7mm round wire	Wedges			
	Bars	Round	Theaded nuts (and intermediate couplers)			
Spiral	Strand	Round	no special device	Zinc or Epoxy Resin	Tight Polyethlene Sheath or none	Grease or Epoxy and/or Paint
	Locked Coil	Shaped and Round	no special device			

* on strand PE sheath may be applied to individual strands

The 'prime' components of the stay system are considered to be essential interdependent features of each proprietary cable-stay system. The approval of a proprietary stay system must be for the particular combination of prime components. For example a proprietary cable system cannot be approved independently of the anchorage or protective system, as the latter components can have a direct effect on the fatigue or corrosion behaviour of the former.

In the case of 'ancillary' components a greater flexibility in approval can be exercised and it should not be necessary to define the exact details of individual clamps, saddles, dampers, etc., which may be used in individual bridges. However, any features which could have an effect on the fatigue or corrosion behaviour must be defined for each proprietary stay system and limits set, beyond which the acceptance certificate would no longer be valid. Examples might be radii of curvature and contact angles of saddles or splay clamps as these can influence local stresses on cable elements. Sealing details at such components must be fully specified where protective sheath is discontinuous. The required mechanical characteristics of dampers may vary from stay to stay or bridge to bridge. However, the limits on contact pressures they exert and any sealing function they may also have, must be fully spelled out in the acceptance certificate.

3.2.2.3 Summary of Scope

In summary, the scope of a specification for fatigue design of bridge stay systems must cover all currently used systems, whether designed by the Client or by the Contractor. The specification must cover the procedure testing to prove that the methods of manufacture of critical components to the required quality is both possible and controllable. This applies particularly to filling of sockets, and the jointing and inspection of sheathing.

The specification must cover the methods of acceptance testing of complete cable systems and define the performance acceptance criteria. This applies both to fatigue and corrosion behaviour.

The specification must cover all necessary controls on manufacture together with testing requirements. These must be designed to be simple and quick to carry out, so that quality control costs are kept to a minimum.

The specification should also require the manufacturers to provide a maintenance schedule for the stays.

3.2.3 Current Specifications

3.2.3.1 Sources of Information

In drafting proposals for the Draft Guide Specification for Fatigue Design of Cables in Cable-Stayed Bridges (given in

Appendix E), the following main sources of information have been consulted.

(i) AASHTO Standard Specification for Highway Bridges (Ref.3)

(ii) PTI Recommendations for Stay Cable Design and Testing 1986 (Ref. 2)

(iii) Manufacturers' Brochures for proprietary systems

(iv) Specifications from bridge Contracts, mainly those from US bridges.

(v) ASTM Standards.

3.2.3.2 <u>AASHTO</u>

The current AASHTO Specification has very little in the way of detailed clauses relevant to cable stay construction. In Division II, Article 4 has clauses on wire specification, grouting, and epoxy bonding, which could be similar to those needed for certain types of stay construction. There are also some general clauses on handling, storage, measurement and payment in Articles 4 and 10 which might be applicable. However, the context in which these clauses were written was clearly not with cable stays in mind. It would therefore seem inappropriate to make reference to these clauses. Clauses dealing with the fabrication of cable stays should preferably be self-contained as are those dealing with expansion joints, railings, etc.

The main reference to the AASHTO specification is for general style, layout, terminology and contract procedure.

3.2.3.3 PTI Recommendations

The Recommendation for Stay Cable Design and Testing by the Post Tensioning Institute (Ref.2) is a most important document in so far as it contains up-to-date proposals for workmanship clauses on a number of commonly used stay systems. It has been drafted particularly with recent US cable-stayed bridges in mind, which mainly use parallel-wire or -strand in cement grouted sheaths. It covers grouted parallel-bar stays, but does not specifically address bridge strand or locked coil strand.

The document has clearly been drafted with the aim of encouraging a technology transfer from post tensioning practice, as used for many years on medium and small span concrete bridges, to stay systems for larger span bridges.

The document has important specification sections. Section 3 on materials makes reference to appropriate ASTMs for wire, strand, bar, sheathing and cement for grout. Section 6 on Test Criteria proposes acceptance fatigue tests on the proposed stay systems for each project. The load bearing appurtenances are to be included (ie anchorages) but it is not clear whether the sheathing and filling agent (usually grout) is to be included in the test. There are details of fatigue acceptance tests on individual elements (wires, strand or bar). Evidence of acceptance of a stay from a prior contract may be submitted for other contracts provided the design is 'similar'. This, of course, is wide open to differing interpretation according to the Engineer's discretion! In view of the expense and potential time delay in approval by acceptance fatigue tests, this could be a cause of serious uncertainty at the time of tender.

Elements of wire, strand or bar are required to be tested for ultimate and fatigue strength both at the time of the original stay acceptance test and also at regular intervals during production for quality control purposes. Specified minimum values are ascribed to both the static tension tests (including UTS, yield stress, Young's Modulus and ductility) and the fatigue tests. For strand a 'one pin' ductility tests is specified, the procedure being spelled out in an Appendix.

The use of specific values for acceptance tests may be somewhat restrictive to designers of proprietary stay systems. For example if galvanised wire or strand is required for increased corrosion protection the minimum specified UTS values of 240 ksi and 270 ksi respectively will not be possible to meet. Table 1 of ASTM A586 Specification for Zinc-coated Parallel and Helical Wire Structural Strand gives a value of 220 ksi.

The use of Bar in accordance with ASTM A722 is advocated by the PTI Document (Ref. 2). This again, is a material developed specifically for prestressing applications where the maximum tension is normally applied at the time of initial stressing. Sizes up to 1 3/8" are covered by ASTM A722. At this thickness in this grade of steel (GUTS = 150 ksi) it would be expected that some control on notch-toughness would be necessary to protect against the possibility of brittle fracture in cold weather. The stipulation in the supplementary bend test (clause S.1) that bends must not be carried out below 61°F (16°C) is an indication of potential brittleness. The presence of secondary stresses and the potential growth of fatigue cracks from threads or corrosion pits (or both), especially in the region of anchorages mean that design detailing and protective systems are particulary critical with this type of tension element.

ASTM A722 has a very low ductility requirement (4% elongation), has few controls on manufacturing methods (including the forming of threads), only has chemical limits on phosphorous and sulphur, has no controls on susceptibility to cleavage fracture, hydrogen embrittlement or stress corrosion cracking and has no mandatory NDT on bars or threads.

It should be noted that any protection system involving the application of heat, corrosive chemicals or electric current to the bar can have potentially detrimental effects on the mechanical properties of this type of material. The use of galvanising, electroplating or flame spraying would, therefore, have to be precluded unless further safeguards were written into the proposals.

Section 8 on corrosion protection is quite short and the only specific requirements are for grouting of steel or polythene pipes, using staged grout lifts. This is clearly a very tricky operation, requiring personnel with considerable experience. It is surprising that this part of the specification is not more detailed. A warning is given on the need to protect the stay prior to grouting, but no recommendations are given as to how this should be done.

Procedure tests are not required to prove that the application of the protection system can be practially done. Obvious candidates for procedure tests are welding of the polythene pipes and the grouting operation. No corrosion acceptance tests are required. There are no requirements for NDT of the final protective systems. This might apply to verification of water tightness of joints and welds in the sheath or pipe and verification of the filling of all voids in the sheath.

Another issue which is not raised is the question of fatigue of the sheath. This applies particularly to the joints which may be welded, threaded or glued. Composite action with the tension elements via the filling agent (grout in particular) will cause stress fluctuations in the sheath.

There is a brief reference to the need for the designer to consider methods of surveillance for detecting corrosion damage during the life of the bridge (see Section 9). However there is no requirement for the Contractor to provide a detailed maintenance schedule to ensure that corrosion damage is prevented. This raises an important point of design principle. It seems vital that there is a 100% confidence in the the long term durabililty of the corrosion protection system at the outset. Reliance on detection of corrosion inside a stay and attempting remedial action to halt or slow down the damage would seem to be a very undesirable policy. This applies particularly to uncoated elements in a grouted sheath.

In summarising this comment on the PTI proposals it must be concluded that the scope of the proposals is too narrow as it stands. The proposals are clearly written around a particular type of stay, for which there has been insufficient detailed knowledge of service performance for a hard and fast specification to be written. The controls proposed for manufacture appear to be inadequate in the areas of procedure testing, NDT and corrosion protection.

3.2.3.4 Manufacturer's Brochures for Proprietary Systems

Brochures by leading manufacturers of stay systems have been a useful source of data on currently used proprietary products. Although many of the systems have common features there are considerable variations in detail, many of which are covered by patents stemming from their development for post-tensioning applications. Brochures by BBR, Freyssinet, Stronghold, and VSL amongst others, are particularly detailed (Refs. 19, 20, 21, & 22) respectively). All these systems have been widely used for cable stayed bridges as well as other structures in the last decade.

It is clear from these brochures that the art of cable stay design, manufacture, erection and protection is still very much in a development stage. This being so, it is important that the ingenuity and experience of individual manufacturers is not hampered by a specification which tends to crystallize the art so that certain types of system are given preferential treatment over others. The fundamental reasons for standardising on engineering products in the past have been to facilitate interchangeability of parts, eg nuts and bolts, and to reduce the costs of stocking an unnecessarily diverse range of almost identical products. In the case of cable stay systems neither of these reasons would appear to be particularly pressing. In more recent times however, the value of standardisation in assuring quality and reliability has been widely appreciated. It is this aspect which is crucial in the case of such an important structural element as a cable stay.

Recent developments of note appear to be increasing interest in using galvanised wire, the use of resin in lieu of zinc in sockets, the use of factory extruded tight fitting sheaths and the use of long lay spiral round wire bundles. Various systems have also been developed for 'softening' the angular change of the cable at the socket by means of flexible tubes and neoprene rings.

The data given in the manufacturer's brochures is primarily intended to give designers information necessarily for incoporation of the proprietary stay system into a particular bridge design. This generally includes leading dimensions and appropriate material specifications for stay elements, anchorage components and sheaths. Mechanical properties of the complete stay are also given. These usually include guaranteed ultimate breaking loads and modulus of elasticity. Fatigue strength data is given to varying degrees. BBR give detailed data for complete stays tests from a number of major contracts. VSL show 2 Wohler curves, one for a single strand and one for a complete cable. Data points are not tabulated or plotted however. Freyssinet similarly gives Wohler curves, but without original data. They also state that the fatigue requirements of British Standard Specification for High Tensile Steel Wire and Strand for the Prestressing of Concrete (BS5896:1980) (which are optional and not mandatory) must be complied with. Stronghold refers to stay fatigue tests but does not give data in any form. The brochures do however give references to reports on fatigue data.

It is important to note that, whilst the brochures do generally emphasise the importance of corrosion protection, no reference is made to any tests having been made on the effectiveness of the protective systems. There is no reference to any requirement for resistance to stress corrosion, which is another optional test in BS5896. BBR do however draw attention to the drastic effect of corrosion on fatigue strength. The various methods of manufacture and installation are described. Detailed material specifications for wire, sockets, socket filler, polyethelene sheathing, and cable fillers such as grout, grease and wax, are given by BBR and Freyssinet. However, none of the manufacturers' specifications include quality control measures for the manufacture or installation of the cables and their protective systems.

In short therefore the manufacturers are very detailed in their provisions for designers to enable proprietary stays to be selected and designed into a bridge. However the data on long term durability and quality control during manufacture is very sparse.

3.2.4 Proposed Guide Specification

The proposed Guide Specification is given in Appendix E. Its preparation has been based on the general principles, outlined in section 3.2.1 above, on the scope discussed in sectiion 3.2.2 above and on current Specifications discussed in section 3.2.3 above.

From the above it is clear that the variety of proprietary cable stay systems available to designers today and the rapidly changing developments in these systems makes it impossible to draft a comprehensive design and manufacturing specification which will ensure satisfactory long term performances for all likely systems at economic cost. The use of new materials, new details and new construction techniques, all with very limited service experience, means that design principles and workmanship standards are not understood to the extent that they are for long-established, well-proven stay systems. It is in fact surprising that radically new systems have been accepted for major designs with so little published research data either from performance tests in the laboratory or monitoring in the field. There can be few cases of radically new designs of major structural members that have been so readily accepted for service in recent years.

One of the major problems in defining a satisfactory specification for the design and manufacture of cable stay systems is the interaction between fatigue and corrosion.

A parallel can be seen here with the problem of defining fatigue strengths for welded offshore structures. In this case extensive environmental corrosion fatigue testing has been carried out to establish safe design rules. Corrosion in welded structural steels has been found to affect fatigue strength by factors not generally in excess of 25% but this is under review and is referred to in a document published by the British Department of Energy entitled "Background to new fatigue design guidance for steel welded joints in offshore structures".In the case of high tensile wires, test have shown that corrosion can easily affect the fatigue strength by a factor of 2, which is an order of magnitude on life. In spite of this, there has been very little attempt to assess the corrosion fatigue resistance of overall cable-stay systems by laboratory testing. This is even more surprising in view of the difference between the typical 20 year design life for offshore structures and the 80 year requirement for bridges in the US.

It is therefore proposed that the Guide Specification should go much further than the recommendations proposed by the P.T.I (Ref. 2) on the subject of acceptance testing. On the other hand detailed guidance is not given on procedures for specific types of cable; for example; Section 8 of the PTI document is very specific to the use of grout in sheathed cables, yet it is not detailed enough to ensure that all operations are adequately controlled.

The principles recommended in the Draft Guide Specification for the Fatigue Design of Cables in Cable-Stayed Bridges are based on current thinking on Quality Assurance. This is that the designer/supplier should work to relevant national standards where they are available.

Beyond this he should be expected, when tendering for a project, to provide full details of the materials, geometry, manufacturing, installation method, quality control procedures and in-service inspection scheme, for the complete system, together with specific test data to satisfy the customer of its long term durability. The method of obtaining the best data must be spelled out by the Guide Specification, so that all cable-stay systems being offered are competing on an equal basis.

When a particular system has been selected on the basis of the supply and maintenance cost and the information tendered above, then production should not be allowed to proceed without trial procedure tests being carried out to prove the more critical manufacturing stages, for example, socketting and inspection of cable fitting components. This is in line with current practice for welding, where welding procedure trials are commonly carried out prior to production welding starting.

Records of quality control inspection throughout the manufacturing and erection stages then would be subject to independent checking by the Client or his agent during the course of manufacture.

The Guide Specification is drafted in a mandatory form with only this basic requirement to be followed for all systems. Its format is similar to that proposed by the P.T.I. but with additional clauses to cover all quality assurance matters.

In conclusion, it should be noted that, in accordance with the brief of this project, the Draft Guide Specification presented in Appendix E covers only the fatigue design of cables in cable-stayed bridges. If sections on static design of cables were added, then the document would become a complete design document for cables in cable-stayed bridges.

CHAPTER 4

CONCLUSIONS AND SUGGESTED FURTHER WORK

4.1 CONCLUSIONS

The aims of this project, concerning the fatigue of cables in cable-stayed bridges, were defined as follows:

(i) to develop criteria and guidelines for fatigue design of cable-stays, and

(ii) to develop practical guidelines for material requirements and for testing wires, strands and cable-stays.

These aims have been achieved through the production of a Draft Guide Specification for the Fatigue Design of Cables in Cable-Stayed Bridges which is presented in Appendix E of this report.

The steps followed in achieving these aims are summarised in the chapters of this report.

Chapter 1 covers the problem statement, research objectives and approach and gives a glossary of terms and definitions.

Chapter 2 covers the gathering and analysis of information. This chapter highlights areas where there is a general shortage of information and data (which is enlarged upon in the second part of this chapter 4). It also gives the background behind the derivation of the loading simulation model and the way this was applied to various computer models of cable-stayed bridges. Aerodynamic effects and local design considerations are described as well as the method of assessing cumulative damage effects. The derivation of the proposed design S-N curve using the existing test data is given.

Chapter 3 is presented in two parts covering the preparation of the cable fatigue design provisions and the preparation of materials and testing requirements. This chapter is in effect a commentary on the Draft Guide Specification for the Fatigue Design of Cables in Cable-Stayed Bridges given in Appendix E.

4.1.1 Guidelines for fatigue design

For the numerical part of the Draft Guide Specification - the guidelines for fatigue design, it has been assumed desirable for the Draft Guide Specification to follow the methods used in the current AASHTO Specification. That is,

(i) to use HS20 truck loading to calculate stress ranges in cables (the lane loading alternative is not considered suitable for this purpose),

(ii) to determine the number of cycles of the calculated stress range to apply (depending on the type of road and expected average daily truck traffic in one direction),

(iii) to determine the allowable design stress range (depending upon the type of cable-stay and whether the structure is redundant or non-redundant),

(iv) compare the calculated stress range from (i) above with the allowable stress range from (iii) above to establish whether or not the design has a fatigue life that is likely to be satisfactory.

Having established the above brief, the basic numerical task that then had to be addressed during this project was to specify the allowable design stress range [item (iii) above] for the different types of cable such that the designer's approach using HS20 trucks outlined above, would accurately reflect the likely damage produced by real traffic flows on real structures. The steps taken in deducing recommendations for these stress categories are given in Section 3.1 of this report.

Several basic decisions and simplifications had to be made in order to keep the end recommendations uniform. In this context, the following points should be noted:-

Fatigue Strength Data

(a) There was insufficient data on large cable tests of each type of cable to be able to draw any significant conclusions on variations of performance between cable types. It was suspected that there was not a large basic difference between the various cable types and this opinion appeared to be justified when the test data was plotted with stress range divided by ultimate stress on one axis. On this plot, the data fitted a reasonably tight distribution with no very obvious trend to separate the performance of any one type of cable from the rest (see Fig. 9). A regression analysis was performed on this data to produce a mean line from which the design line was deduced. No useable data for steel bars was available and hence no recommendations have been made for allowable design stress range for stay systems using parallel bars.

(b) When the test data were plotted, it became clear that the design S-N curve had a logarithmic slope close to four which is flatter than the slope (close to three) adopted for the stress categories in the AASHTO Specification used. It was decided not to use the recent stress categories from AASHTO, because this would necessarily incorporate additional conservatism. New categories were produced for the different cable types and allowable design stress ranges calculated for each.

(c) No obvious mean stress effect was observed in the available test data for large cables (compare Figures 7 and 8). This had not been expected and therefore permitted a simplification, in that no factor for this effect was required.

Fatigue Loading

(d) It was found that there was a relatively small scatter in fatigue damage noted between the various posssible cable locations in cable-stayed bridges. In particular, it had been expected that there would be a marked difference between back-stay and all other fore-stays. From the computer modelling, it was found that this difference did exist, but was smaller than expected. It was therefore decided not to differentiate between cable locations and to present results as a band of values applicable to all cable locations.

(e) It was found that, despite predictions, there appeared to be no significant span effect. The length of the span did not noticeably effect the damage felt by the cable-stays. The reason for this seemed to be that the major contributions to fatigue damage were the bunches of trucks which were so spaced that only one bunch was carried by a bridge at any one time (for spans between 400 ft and 1500 ft). This permitted further simplification.

(f) Table 10.3.2A of the AASHTO Specification gives the number of cycles of truck loading to be applied depending on the expected ADTT level. It was found necessary to alter the threshold ADTT level from 2500 to 900 for the change in applied number of cycles from 500,000 to 2,000,000, in order to maintain a consistently uniform set of factors. This point is detailed further in Section 3.1 of this report.

(g) It is argued (in Section 3.1), that when the methods of the present AASHTO Specification are adopted, there is no basic difference in performance of cable-stays in single plane or twin plane configurations.

4.1.2 Guidelines for Material Requirements and Testing

It was concluded that, with the large range of proprietary cable-stay products now available, and with this number likely to grow in the near future, the time was right to suggest a formalization of the situation in order to ensure adequate product performance. It has been suggested that a series of tests for each new product should be set up firstly through a series of procedure tests to ensure that the manufacturing techniques are successful, and secondly to show, through acceptance tests, that the product can achieve the desired performance. In addition to this, it is suggested that a series of production tests should be instigated to ensure that the quality of the product is maintained throughout production.

These tests have addressed the additional question of corrosion since, although not directly within the brief of fatigue design, the effect of corrosion on fatigue performance can be so significant that it cannot be ignored.

This thinking is expanded upon in Section 3.2 of this report.

4.1.3 Summary of Basic Conclusions

The aims of this project have been achieved through the production of the Draft Guide Specification for Fatigue Design of Cables in Cable-Stayed Bridges (presented in Appendix E). It should be noted that this document has been produced from a relatively limited amount of data and could well be much refined and improved upon when more data is available. There is a need for the suggested further work described in the following section as well as feedback on the use of the Draft Guide Specification itself. In particular, it should be remembered that three crucial assumptions have been made during production of the Draft Guide Specification:

(i) It has been assumed that no corrosion takes place during the life of the cable-stays. It is therefore extremely important to develop satisfactory corrosion protection systems if the stress levels given here are to be used reliably.

(ii) It has been assumed that no appreciable wind effects are inflicted upon the cable-stays. It is therefore important to provide suitable damping mechanisms if they are so required.

(iii) It has been assumed that, if the cable-stay design should eventually prove to be unable to fulfil its required life span, it should be possible to replace each cable in the bridge without inconvenience to traffic flow or detrement to neighbouring cables. It is therefore important to adopt a design configuration that will permit such replacement.

The manufacturers of cable-stay systems are continually developing their products as well as introducing new methods and ideas. This state of affairs should be encouraged in order to enlarge the available choice of systems and (through development) reduce any shortcomings that may exist. However, there can be a tendency to utilise new and attractive ideas while, to some extent, detracting from the benefits of tried and tested methods of production. This could have unfortunate consequences if allowed to progress into modern bridges before exhaustive testing has been conducted on the newer systems.

4.2 SUGGESTED FURTHER WORK

It was noted in Section 4.1 above that the Draft Guide Specification for the Fatigue Design of Cables in Cable-Stayed Bridges has been produced from a limited amount of data. During production of this report, it became clear that several areas would benefit from further research work, which in turn would yield more data and permit refinement and improvement to the recommendations made in this report.

The majority of suggested further work can be summarised under two broad subjects. The first concerning records of past and present performance with, maybe, predictions for future performance. The second concerning detail design of specific items.

4.2.1 Further Work on Obtaining Records Of Performance

4.2.2.1 Existing Bridges

The literature search did not reveal any records of cable stress history on any real bridges which could be related directly to traffic flow. Research into actual stress histories on actual bridges would be extremely valuable.

This researach would also reveal any difference in fatigue damage felt by single planes of stays compared with twin planes of stays. It would also highlight variations in fatigue damage experienced between back-stay and fore-stay cables.

In addition to this, a more general research program into the overall performance of stay cable systems on existing bridges is recommended. This might take the form of the world wide survey referred to earlier, but with a brief to interview each bridge owner with an aim to answer all of the questions listed in the questionnaire form sheets enclosed in Appendix B of this report.

4.2.1.2 Fatigue Tests

More full size cable tests to add more points to the data base of S-N information. These tests should follow the suggestions for acceptance tests outlined in the Draft Guide Specification. The data on bars is particularly sparse at present. If the system calls for a blocking agent (grease or grout, etc.), then this should be included in the test specimens.

It seems also desirable to persue research into extrapolation of small scale tests (single wire/strand) with a view to predicting behaviour of full size cables.

4.2.1.3 Truck Traffic

Further research into the spectrum of vehicle weights would be desirable, especially if estimates of likely increases in truck weights can be made and spectrums for different classes of highway can be drawn up.

In particular, work on this project highlighted the paucity of data on headway patterns. The figures used in this report are not well founded and should be refined when better, more accurate information becomes available.

4.2.1.4 Comments on the Draft Guide Specification

The Draft Guide Specification produced in this report is based on performance principles not currently in common use and as such, is bound to benefit from refinement and possible improvements when experience in its use has been attained.

4.2.2 Further Work on Detail Design of Specific Items

4.2.2.1 Waterproofing

It seems crucial to research into methods of confidently preventing ingress of water into cable-stay systems. At present, most of the systems offered on the market have some shortfalls in this area.

4.2.2.2 Galvanizing

There has been a noticeable reluctance in some places to use galvanizing as a means of corrosion protection. The reasons have been given as a lower cable breaking load and also allegations of possible hydrogen embrittlement. The reduction in breaking load is generally small and such a loss in ultimate strength would seem to be a small price to pay for the greatly enhanced corrosion protection that results with a galvanized product. The allegations of possible hydrogen embrittlement have not been substantiated by any examples found in the literature. Research into present galvanizing techniques (particularly in the United States) and likely effects on cable-stay systems should serve to allay any doubts on the benefits that can be gained from the use of properly controlled galvanizing.

4.2.2.3 Socketing

There is some indication in the literature that, for zinc socketing, the temperature at which the zinc is poured can have an effect on the fatigue life. The data presented so far is based on small scale tests, and some of the conclusions drawn seem unrealistic in some respects. Further research would be beneficial.

The literature also highlights some doubts about resin socketing. Research into micro-cracking and its effects on concentration of corrosion risk should be carried out.

4.2.2.4 Use of Non-Destructive Testing (NDT)

There is a system of NDT currently used in the mining industry (Refs 174 and 175) which can "see" corrosion as well as broken wires. This makes use of the "Hall effect" and should be researched further for its application to larger cables and its ability to reach close up to anchorages. It is understood that there is now a development of this system on the market that has been used on larger cables.

The documentation did not reveal any NDT method that claimed to be wholly successful within, and adjacent to, anchorages zones. This aspect of NDT should be researched further. Possible approaches considered by researchers have included the use of accoustic emission or accoustic pulsing methods.

4.2.2.5 Redundant Structures

Research into the behaviour of redundant structures. There is a need for guidance for the designer on deciding whether the cables in question will behave in a redundant manner (ie will they off load safely to adjacent cables in the event of sudden failure of one cable which otherwise could lead to progressive collapse).

4.2.2.6 Bending Effects

There are several theoretical suggestions in the literature to cover bending effects in free cable lengths. In particular, there seems to be a need to measure real effects on real cables, particularly in the region of anchorages and saddles.

4.2.2.7 Damping Mechanisms

It would be beneficial to produce a designer's guide on the application of dampers to cable systems and their likely effects.

REFERENCES

It should be noted that a full bibliography is given in Appendix C arranged in alphabetical order by author or source.

1. German Code DIN1073. Steel Road Bridges, Design Base July 1974

2. Post-Tensioning Institute. Recommendations for Stay Cable Design and Testing. January 1986 and February 1990.

3. AASHTO. Standard Specifications for Highway Bridges 1983 (Plus modifications 1984, 1985)

4. GUTE, PODOLNY, SEIM, TANG, ULSTRUP, YIU. Bibliography and Data on Cable-Stayed Bridges. ASCE Proc; V. 103, N. ST10 October 1977.

5. GOODYEAR. Issues in Stay Cable Fatigue Design Loading. CIB Major Bridge Conference New York April 1986.

6. FEDERAL HIGHWAY ADMINISTRATION. Loading Spectrum Experienced by Bridge Structures in USA. Report FHWA/RD-85/012 Feb. 1985

7. FEDERAL HIGHWAY ADMINISTRATION. Report FHWA/OH-83/001

8. BRITISH STANDARDS INSTITUTE. BS5400 Steel, Concrete and Composite Bridges. Part 10:1980 Code of Practice for Fatigue.

9. Conference on Bridge Design Rules. Bridge Aerodynamics 25-26 March 1981.

11. G.P. TILLY. Design and Performance of Stay Cables. TRRL Paper to Czechoslovak Scientific and Technical Society. October 1986.

12. AIMONE. Non-destructive Wire Rope Tester. October 1986

13. HOBBS, SMITH. Fatigue Performance of Socketed Terminations to Structural Strands. ICE Proceedings Pt. 2 Paper 8605 March 1983.

14. FLINT & NEILL PARTNERSHIP. Investigation of Fatigue Failure in Socketed Terminations on Structural Cables. April 1980.

15. T.R. GURNEY. Fatigue Tests to Assess the Validity of Miner's Rule. Proceedings of Royal Society London 1983.

16. T.R. GURNEY. Fatigue Tests on Fillet Welded Joints under Simple Variable Amplitude Loading. T.W.I. Research Report 182/1982 May 1982.

17. T.R. GURNEY. Fatigue Tests under Variable Amplitude Loading. T.W.I. Research Report 220/1983 July 1983.

18. T.R. GURNEY. Fatigue Tests on Fillet Welded Joints under Variable Amplitude Loading. T.W.I. Research Report 293. November 1985.

19. BBR. Cable Stayed Structures. 1986

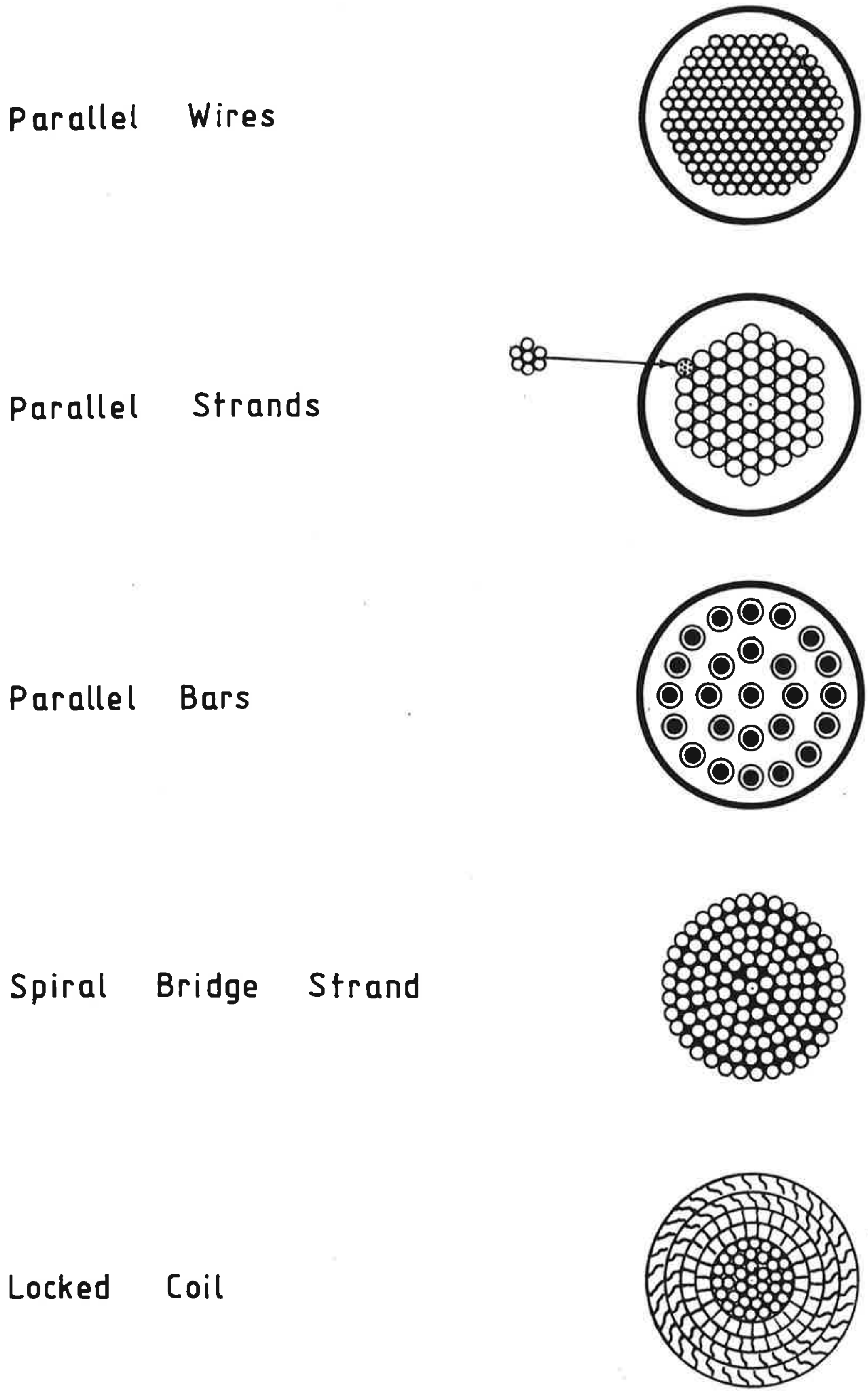
20. FREYSSINET Various unpublished test results 1980 and 1981

21. STRONGHOLD. Cable-Stayed Bridges. Publication No. 54. January 1986.

22. VSL INTERNATIONAL LOSINER LTD. VSL Stay Cables for Cable-Stayed Bridges. January 1984.

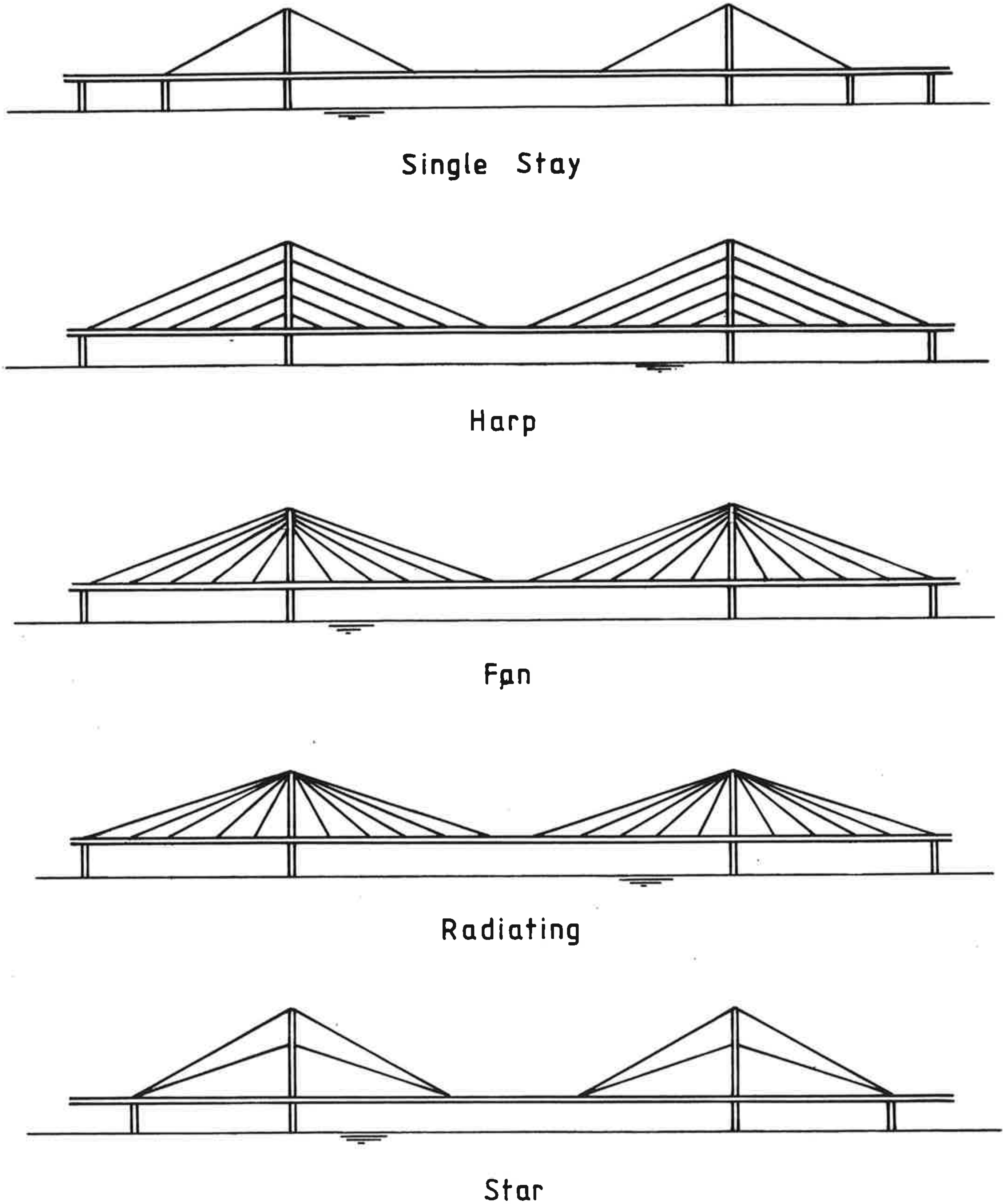
FIGURES

LIST OF FIGURES



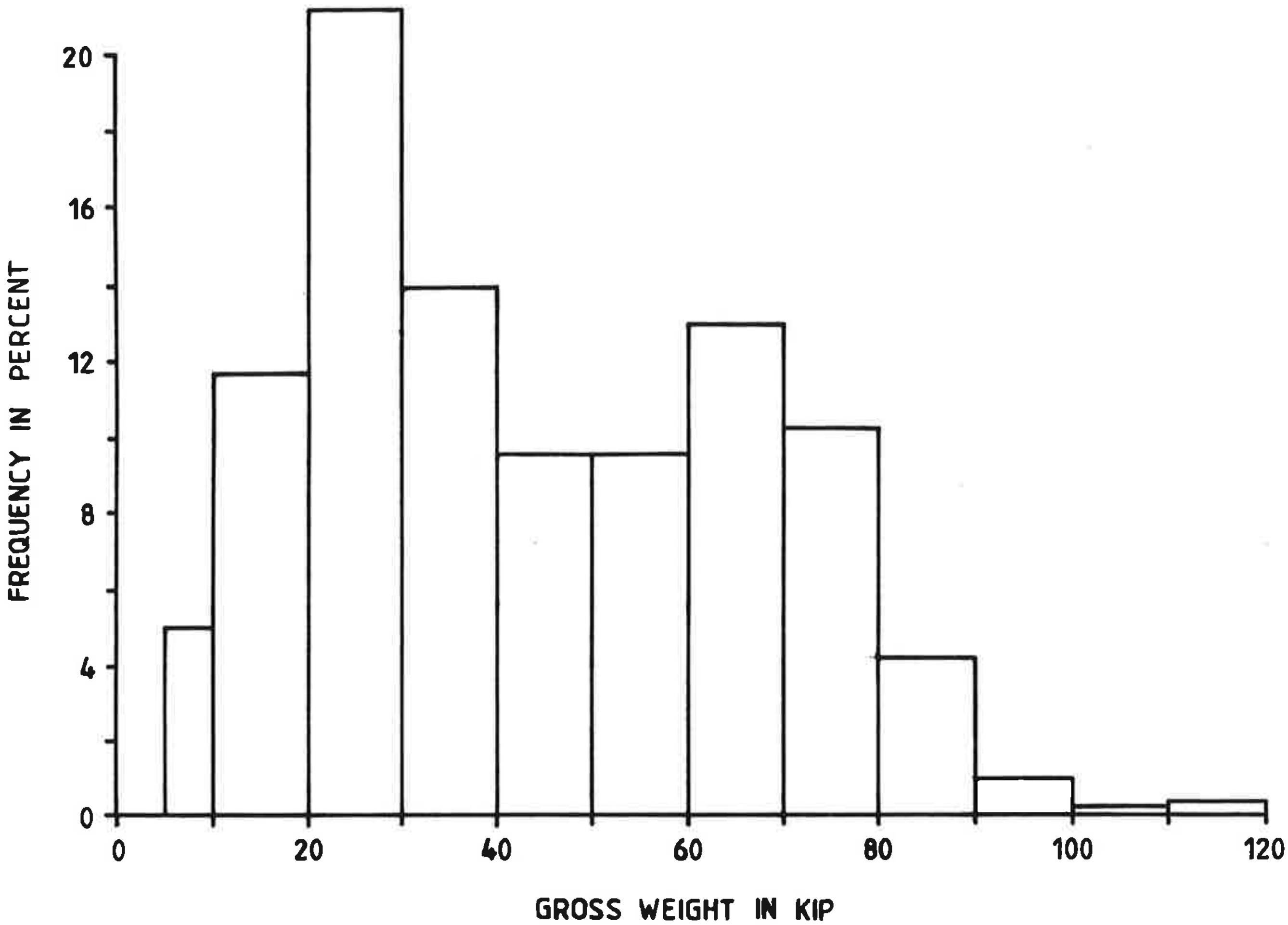
CROSS SECTIONS OF SOME STAY CABLES

FIG. 1



BASIC ARRANGEMENTS OF STAY CABLES

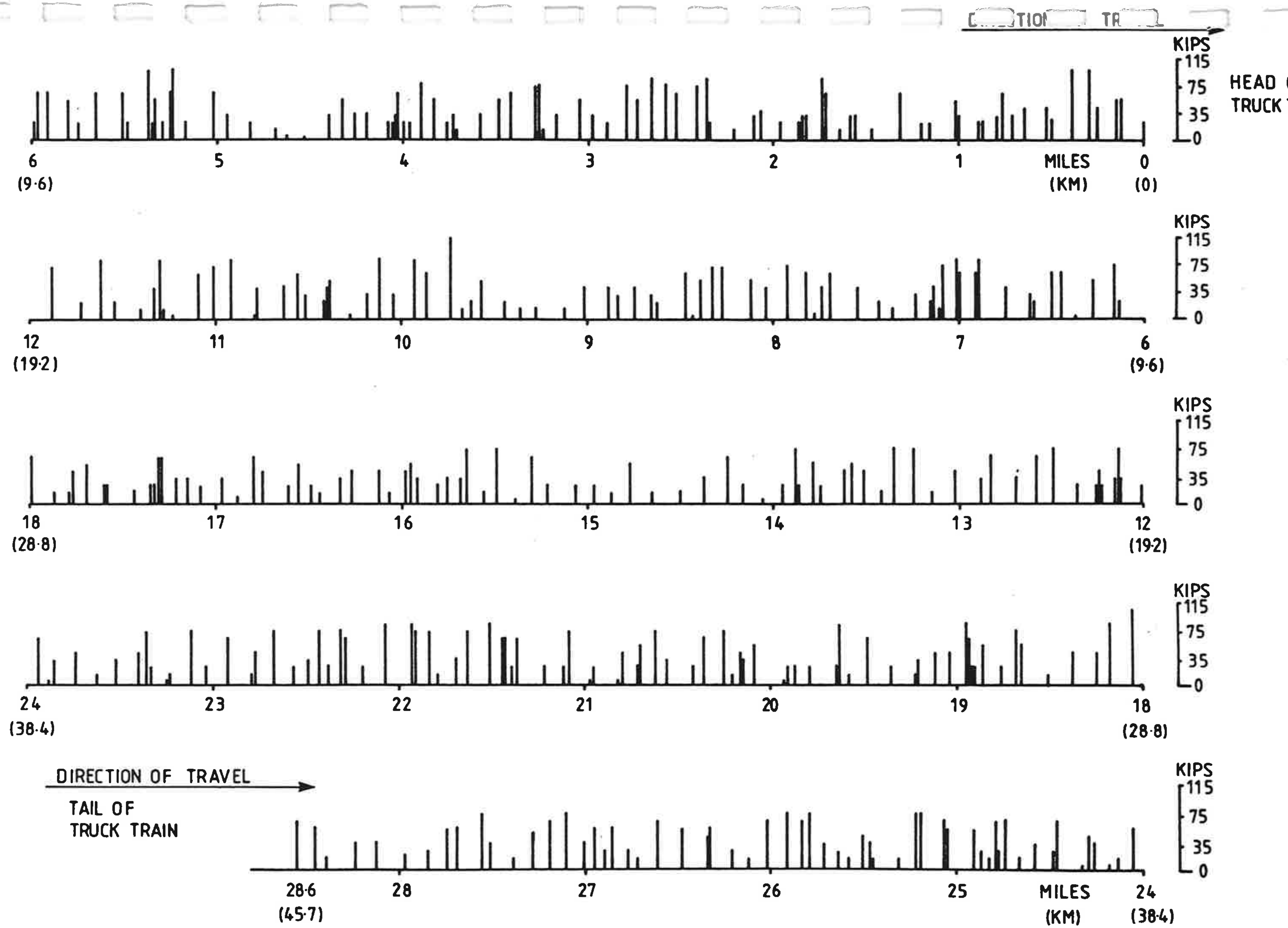
FIG. 2



Original data taken from FHWA National Survey
(27513 Trucks at 32 Sites)
and modified for use in this report.

TRUCK GROSS WEIGHT HISTOGRAM

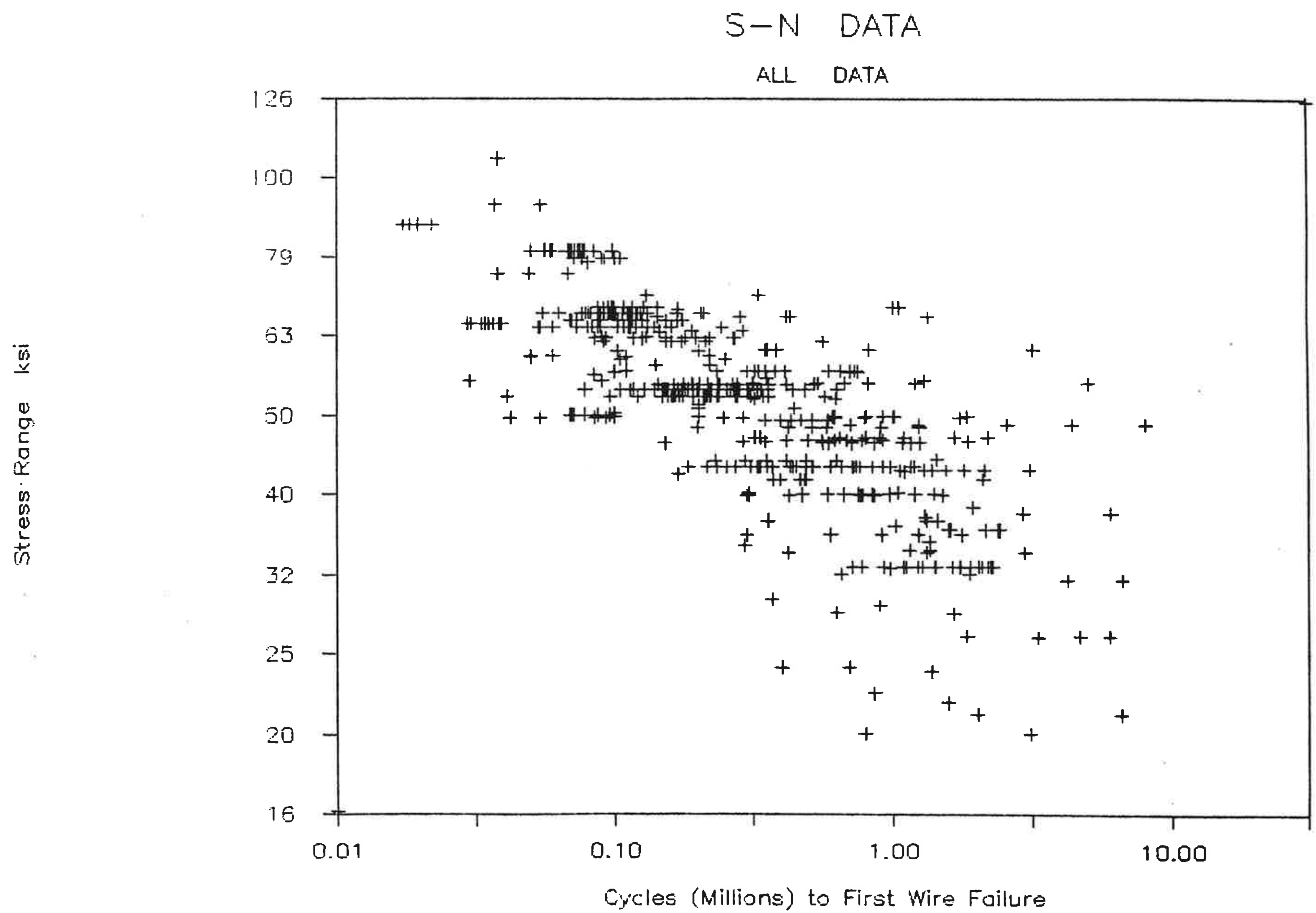
FIG. 3



TYPICAL TRUCK TRAIN LOAD HISTORY

FIG. 4

FIG.4



S-N DATA - ALL TEST RESULTS AS PUBLISHED

FIG.5

FIG.5

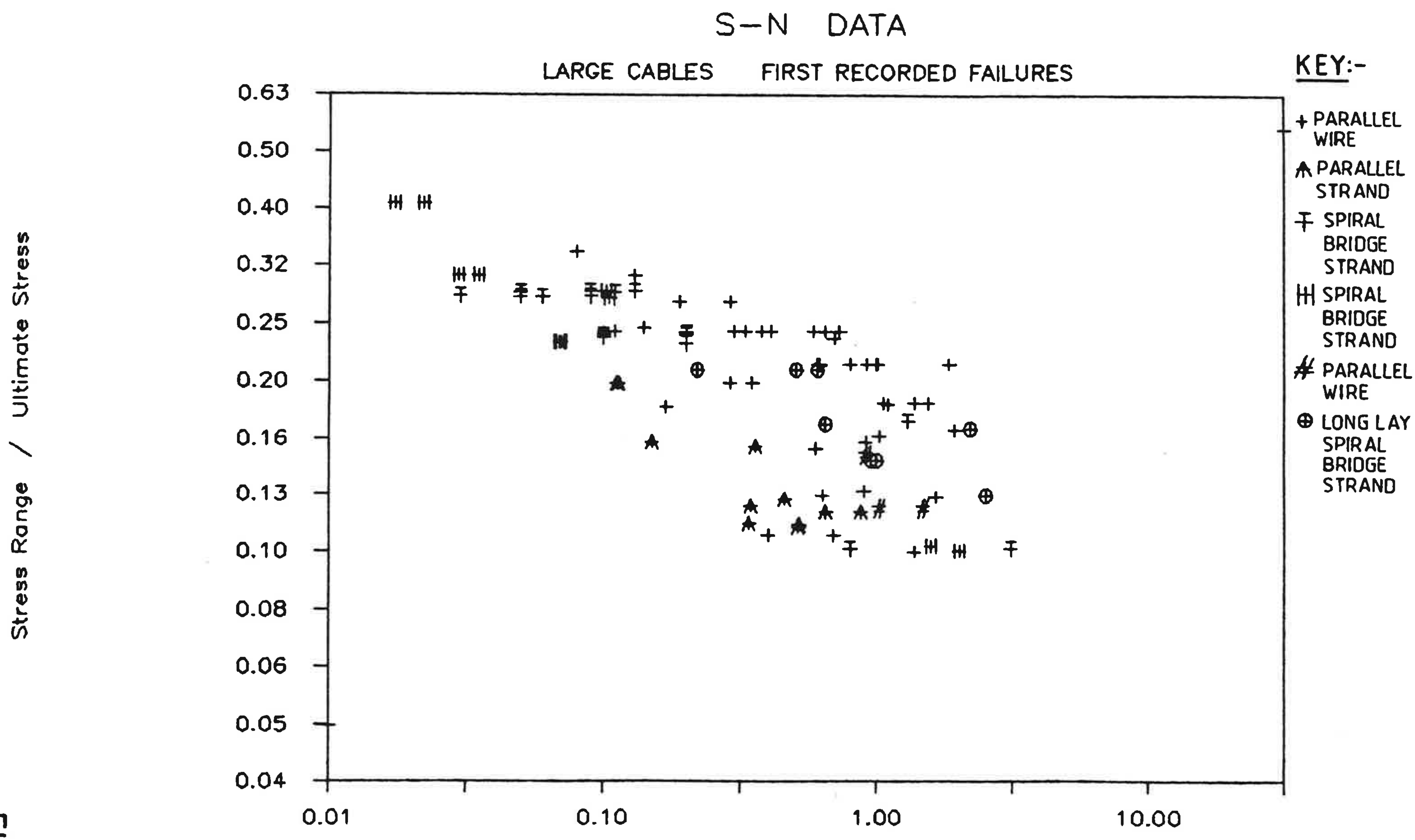
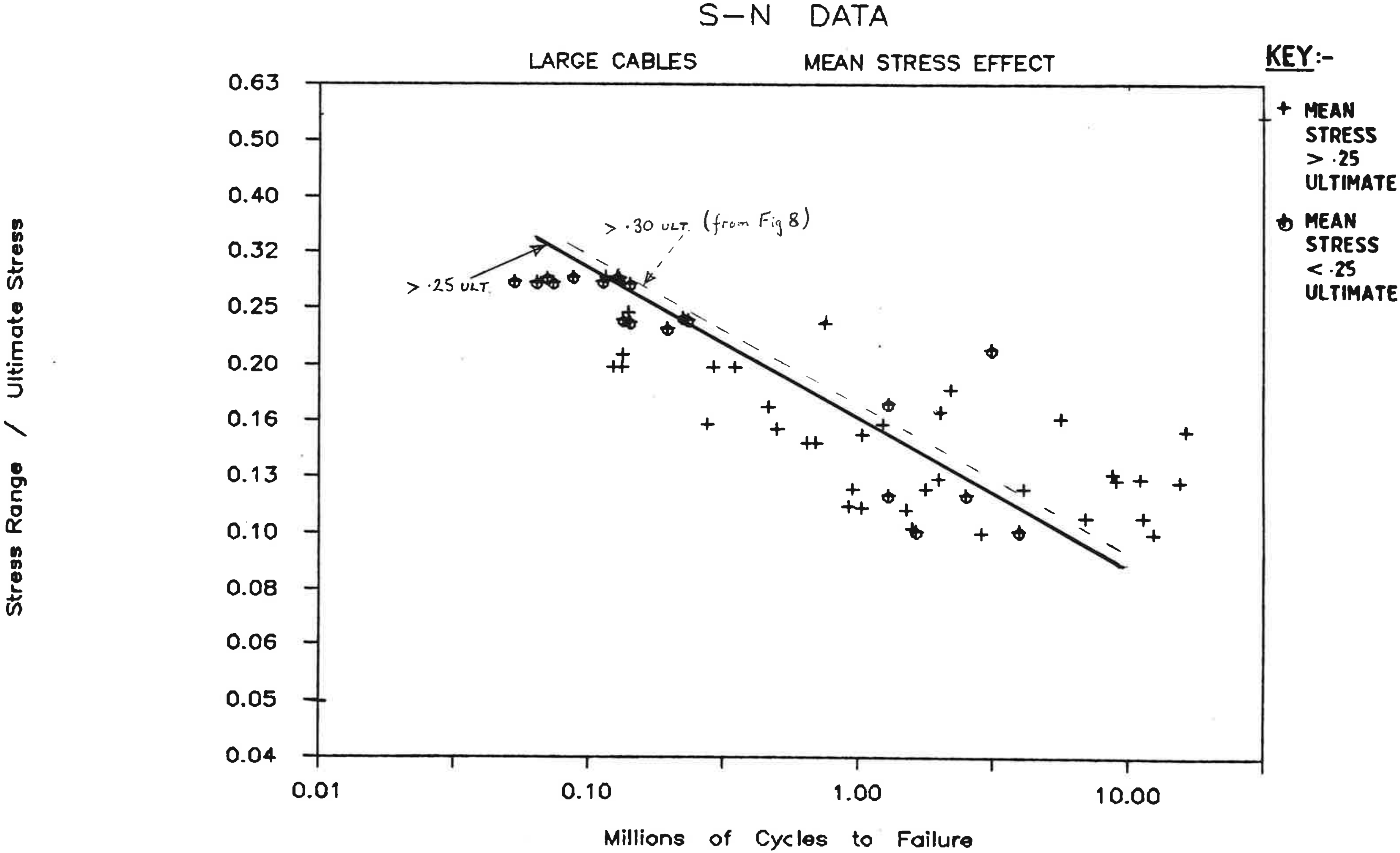


FIG.6

S-N DATA - LARGE CABLE TEST RESULTS AS PUBLISHED

FIG.6



S-N DATA - ILLUSTRATION OF MEAN STRESS EFFECT AT 25% ULTIMATE FIG.7

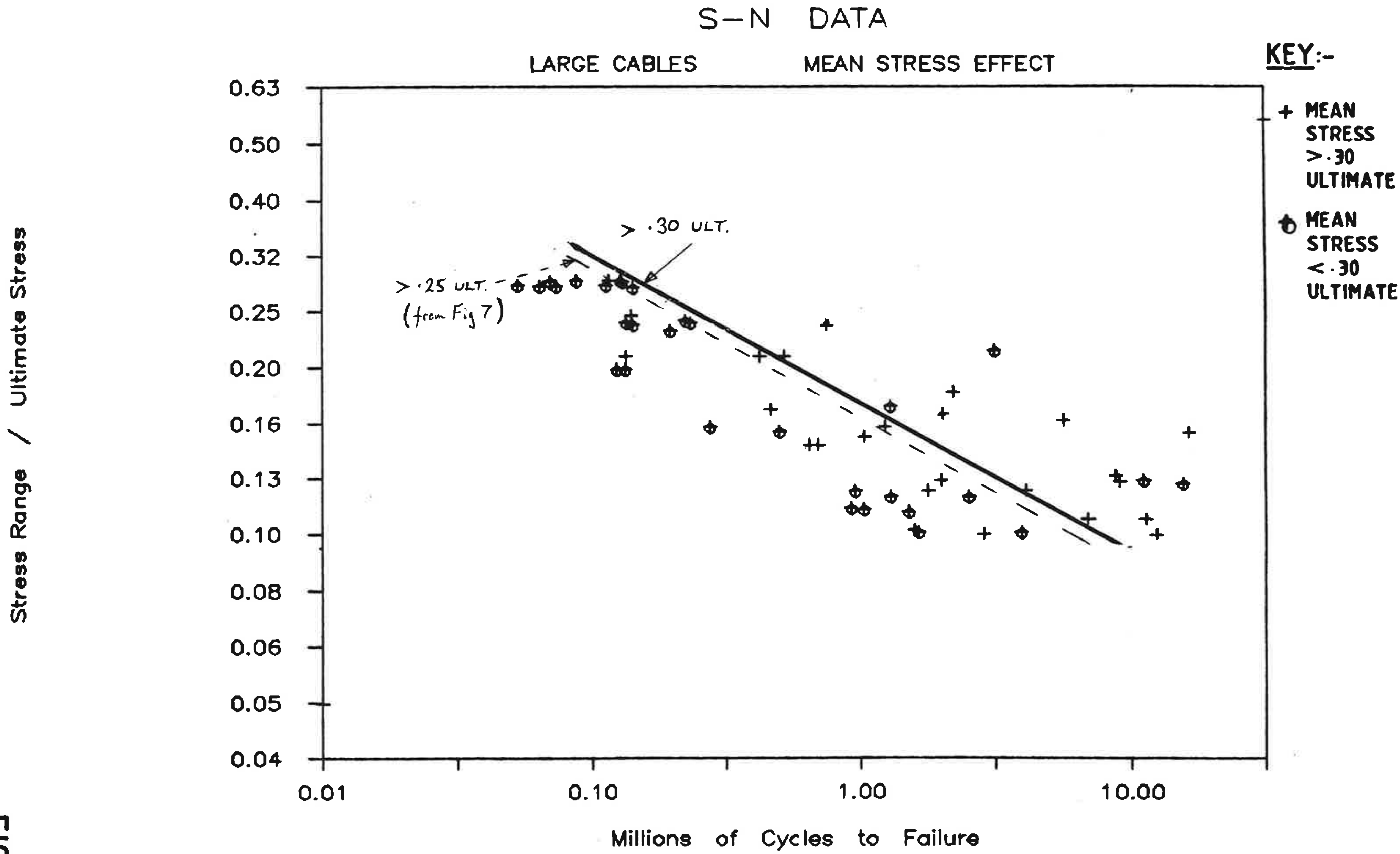
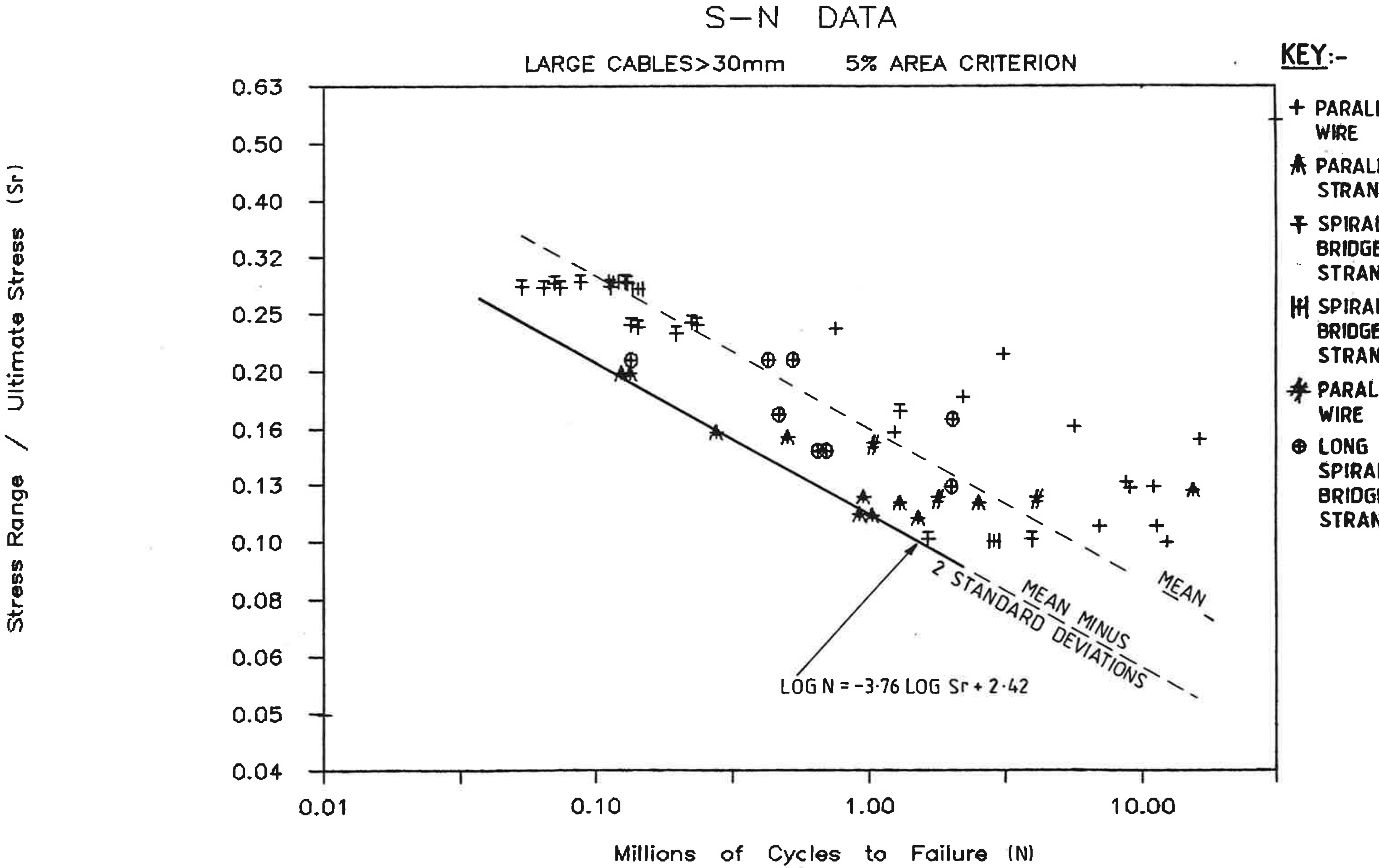


FIG.8 S-N DATA-ILLUSTRATION OF MEAN STRESS EFFECT AT 30% ULTIMATE FIG.8



S-N DATA - FOR CABLES OF DIAMETER GREATER THAN 30mm DERIVATION OF DESIGN CURVE

FIG 9

FIG

Y = % ULTIMATE		Stress Range		MEAN STRESS	Ultimate Stress		Stress Range	Recorded failure		SUM	SUM	SUM	SUM	SUM	SUM
										-40.941	312.482	32.908	1869.715	-246.208	53
Source	Cable Type	MPa	ksi	% ULTIMATE	Mpa	ksi	% Ultimate	Millions of Cycles	%Area Cable	RANGE OF LINES USED = 8..60					
			***			***	"Y"	"Cyc"							
	Scaling		16.0				0.050	0.01		"Y"	"X"	"Y*Y"	"X*X"	"X*Y"	"N"
	Scaling		125.0				0.550	31.60		LOG(Y)	LOG(Cyc)				
BBR 29#	127No7mm	344	49.9	0.143	1618	234.6	0.213	3.09	5.00	-0.6724	6.4900	0.4521	42.1196	-4.3640	1
BBR 43#	241No7mm	291	42.2	0.360	1618	234.6	0.180	2.19	5.00	-0.7451	6.3404	0.5552	40.2012	-4.7242	2
BBR 42#	241No7mm	245	35.5	0.374	1618	234.6	0.151	16.27	5.00	-0.8198	7.2114	0.6721	52.0041	-5.9120	3
BBR 31#	295No7mm	196	28.4	0.288	1570	227.6	0.125	11.01	5.00	-0.9036	7.0418	0.8166	49.5868	-6.3633	4
BBR 34#	295No7mm	167	24.2	0.303	1570	227.6	0.106	6.96	5.00	-0.9732	6.8426	0.9471	46.8213	-6.6591	5
BBR 41@	127No7mm	251	36.4	0.371	1570	227.6	0.160	5.60	5.00	-0.7962	6.7482	0.6340	45.5380	-5.3731	6
BBR 33@	295No7mm	200	29.0	0.303	1570	227.6	0.127	8.68	5.00	-0.8949	6.9385	0.8008	48.1431	-6.2091	7
BBR 40@	127No7mm	195	28.3	0.398	1570	227.6	0.124	9.00	5.00	-0.9059	6.9542	0.8206	48.3615	-6.2996	8
BBR 32@	295No7mm	167	24.2	0.303	1570	227.6	0.106	11.30	5.00	-0.9732	7.0531	0.9471	49.7459	-6.8639	9
BBR 36@	121No.25	165	23.9	0.350	1654	239.8	0.100	12.33	5.00	-1.0011	7.0910	1.0021	50.2818	-7.0984	10
BBR 30*	85No7mm	245	35.5	0.372	1570	227.6	0.156	1.23	5.00	-0.8070	6.0899	0.6513	37.0869	-4.9148	11
BBR 35*	83No.25	392	56.8	0.332	1656	240.1	0.237	0.75	5.00	-0.6258	5.8751	0.3916	34.5163	-3.6765	12
TRRL1/1?	63 Spiral	138	20.0	0.225	T		0.101	1.64	5.00	-0.9949	6.2135	0.9897	38.6078	-6.1816	13
TRRL1/3?	63 Spiral	138	20.0	0.225	T		0.101	3.94	5.00	-0.9949	6.5949	0.9897	43.4933	-6.5610	14
TRRL1/2?	63 Spiral	382	55.4	0.225	T		0.280	0.05	5.00	-0.5532	4.7243	0.3060	22.3188	-2.6135	15
TRRL1/5?	63 Spiral	382	55.4	0.225	T		0.280	0.11	5.00	-0.5532	5.0531	0.3060	25.5336	-2.7954	16
TRRL3/3?	62 LkCoil	348	50.5	0.225	T		0.238	0.14	5.00	-0.6235	5.1553	0.3887	26.5775	-3.2143	17
TRRL4/1?	53 Spiral	345	50.0	0.226	T		0.240	0.23	5.00	-0.6198	5.3692	0.3841	28.8285	-3.3278	18
TRRL4/3?	53 Spiral	345	50.0	0.226	T		0.240	0.13	5.00	-0.6198	5.1271	0.3841	26.2872	-3.1777	19
TRRL4/2?	53 Spiral	409	59.3	0.226	T		0.284	0.07	5.00	-0.5460	4.8451	0.2981	23.4750	-2.6454	20
TRRL4/4?	53 Spiral	409	59.3	0.226	T		0.284	0.13	5.00	-0.5460	5.1139	0.2981	26.1524	-2.7922	21
TRRL5/1?	44 Spiral	357	51.8	0.222	T		0.242	0.22	5.00	-0.6154	5.3483	0.3787	28.6044	-3.2915	22
TRRL5/2?	44 Spiral	411	59.6	0.222	T		0.279	0.07	5.00	-0.5547	4.8692	0.3077	23.7094	-2.7011	23
TRRL5/4?	44 Spiral	411	59.6	0.222	T		0.279	0.06	5.00	-0.5547	4.8062	0.3077	23.0994	-2.6661	24
TRRL7/3?	36 Spiral	258	37.4	0.228	T		0.170	1.29	5.00	-0.7705	6.1106	0.5936	37.3393	-4.7080	25
TRRL7/1?	36 Spiral	353	51.2	0.228	T		0.232	0.20	5.00	-0.6342	5.2923	0.4023	28.0080	-3.3566	26
TRRL7/2?	36 Spiral	434	62.9	0.228	T		0.286	0.09	5.00	-0.5441	4.9445	0.2960	24.4479	-2.6901	27
TRRL7/4?	36 Spiral	434	62.9	0.228	T		0.286	0.13	5.00	-0.5441	5.1072	0.2960	26.0836	-2.7787	28
HOBBS-1#	38 Spiral	146	21.2	0.350	H92	No wires	0.100	2.86	5.00	-1.0000	6.4561	1.0000	41.6807	-6.4561	29
HOBBS-2#	38 Spiral	407	59.0	0.239	H92	No wires	0.278	0.14	5.00	-0.5560	5.1553	0.3091	26.5775	-2.8661	30
HOBBS-3#	38 Spiral	417	60.5	0.443	H92	No wires	0.285	0.12	5.00	-0.5452	5.0645	0.2972	25.6487	-2.7609	31
ANDRA&Z#	63No.7mm	245	35.5	0.356	#1638	237.5	0.150	1.03	5.00	-0.8251	6.0137	0.6809	36.1643	-4.9622	32
ANDRA&Z#	78No.7mm	196	28.4	0.371	#1638	237.5	0.120	1.78	5.00	-0.9221	6.2499	0.8502	39.0616	-5.7628	33
ANDRA&Z#	77No.7mm	196	28.4	0.371	#1638	237.5	0.120	4.09	5.00	-0.9221	6.6121	0.8502	43.7205	-6.0968	34
AND+S8#	85No15.75	202	29.3	0.295	A1638	237.5	0.123	15.40	5.00	-0.9090	7.1875	0.8262	51.6605	-6.5332	35
AND+S9#	85No15.75	198	28.7	0.273	A1775	257.4	0.112	0.92	5.00	-0.9525	5.9624	0.9073	35.5498	-5.6794	36
AND+S10#	85No15.75	197	28.6	0.271	A1775	257.4	0.111	1.02	5.00	-0.9547	6.0103	0.9115	36.1237	-5.7382	37
AND+S11#	31No15.75	207	30.0	0.233	A1775	257.4	0.117	2.51	5.00	-0.9332	6.3995	0.8709	40.9536	-5.9722	38
AND+S12#	31No15.75	207	30.0	0.233	A1775	257.4	0.117	1.29	5.00	-0.9332	6.1103	0.8709	37.3352	-5.7023	39
AND+S13#	31No15.75	195	28.3	0.274	A1775	257.4	0.110	1.50	5.00	-0.9592	6.1770	0.9200	38.1548	-5.9247	40
AND+S16#	19No15.75	251	36.4	0.252	A1638	237.5	0.153	0.50	5.00	-0.8146	5.6998	0.6636	32.4882	-4.6433	41
AND+S17#	19No15.75	324	47.0	0.260	A1638	237.5	0.198	0.12	5.00	-0.7038	5.0934	0.4953	25.9429	-3.5846	42
AND+S19#	19No15.75	255	37.0	0.281	A1638	237.5	0.156	0.28	5.00	-0.8078	5.4393	0.6525	29.5863	-4.3937	43
AND+S21#	19No15.75	324	47.0	0.260	A1638	237.5	0.198	0.13	5.00	-0.7038	5.1239	0.4953	26.2539	-3.6060	44
AND+S28#	55No15.75	196	28.4	0.299	A1638	237.5	0.120	0.95	5.00	-0.9221	5.9795	0.8502	35.7550	-5.5135	45
BRIDON1?	37No.7.1m	316	45.8	0.313	D1520	220.4	0.208	0.13	5.00	-0.6822	5.1271	0.4653	26.2872	-3.4975	46
BRIDON2?	37No.7.1m	316	45.8	0.313	D1520	220.4	0.208	0.42	5.00	-0.6822	5.6274	0.4653	31.6672	-3.8387	47
BRIDON3?	37No.7.1m	316	45.8	0.313	D1520	220.4	0.208	0.52	5.00	-0.6822	5.7193	0.4653	32.7108	-3.9015	48
BRIDON4?	37No.7.1m	254	36.8	0.313	D1520	220.4	0.167	0.47	5.00	-0.7770	5.6675	0.6037	32.1200	-4.4037	49
BRIDON5?	37No.7.1m	250	36.3	0.313	D1520	220.4	0.164	2.00	5.00	-0.7839	6.3017	0.6145	39.7112	-4.9399	50
BRIDON6?	37No.7.1m	220	31.9	0.313	D1520	220.4	0.145	0.65	5.00	-0.8394	5.8109	0.7046	33.7666	-4.8778	51
BRIDON7?	37No.7.1m	220	31.9	0.313	D1520	220.4	0.145	0.70	5.00	-0.8394	5.8432	0.7046	34.1434	-4.9049	52
BRIDON8?	37No.7.1m	190	27.5	0.313	D1520	220.4	0.125	1.99	5.00	-0.9031	6.2991	0.8156	39.6783	-5.6886	53

NOTE :

" - " = First failure where at least 2 failures were recorded during test period

" * " = 2nd. or later failure where at least 2 failures were recorded .

Where "Source" shows no " - " or " * " , then only one failure was recorded

" # " = Interpolated from two recorded results

" @ " = Interpolated from a single result using other results

Key to data sources:-

BBR	: Ref. 177
TRRL	: Unpublished
HOBBS	: Ref. 15
ANDRA + Z	: Ref. 197
ANDRA + S	: Ref. 196
BRIDON	: British Ropes unpublished

RANGE OF LINES USED = 8..60 TOTAL NUMBER = 53

PERPENDICULAR REGRESSION	VERTICAL REGRESSION		HORIZONTAL REGRESSION	
!! Ixx =1.282622				
!! Iyy =27.36113	SUMXY-(SUMX*SUMY/N)		SUMXY-(SUMX*SUMY/N)	
!! Ixy =-4.82655		-4.82655		-4.82655
!!	SUMX*X-(SUMX*SUMX/N)		SUMY*Y-(SUMY*SUMY/N)	
!!ALPHA =-1.39353		27.36113		1.282622
!! RADIANS	SLOPE b =		SLOPE b =	
!!		-0.17640		
* SLOPE = -5.582		-5.66887		-3.76303
	INTERCEPT ON Y AXIS			
	LOG(Y) =	0.267579		
	STRESS =	1.851736		
INTERCEPT ON X AXIS	INTERCEPT ON X AXIS		INTERCEPT ON X AXIS	
LOG(X) =1.583770	LOG(X) =	1.516872		=2.989064
			LOG(N) LOG(Sr)	
X = + m Y + C	X = + m Y + C		X = + m Y + C	
-5.582 1.584	-5.669	1.517	-3.763	2.989

!!
!!X1bar =1.895878
!!X2bar =1.104121
!!
Yint104=-0.43816
!! Y =0.364618

## mm= -227.2 ##	LG126-L16	0.89
Yint107=-0.96715	Y104-LG16	-1.6
!! Y =0.107855	Y107-LG16	-2.1

FIG.9B

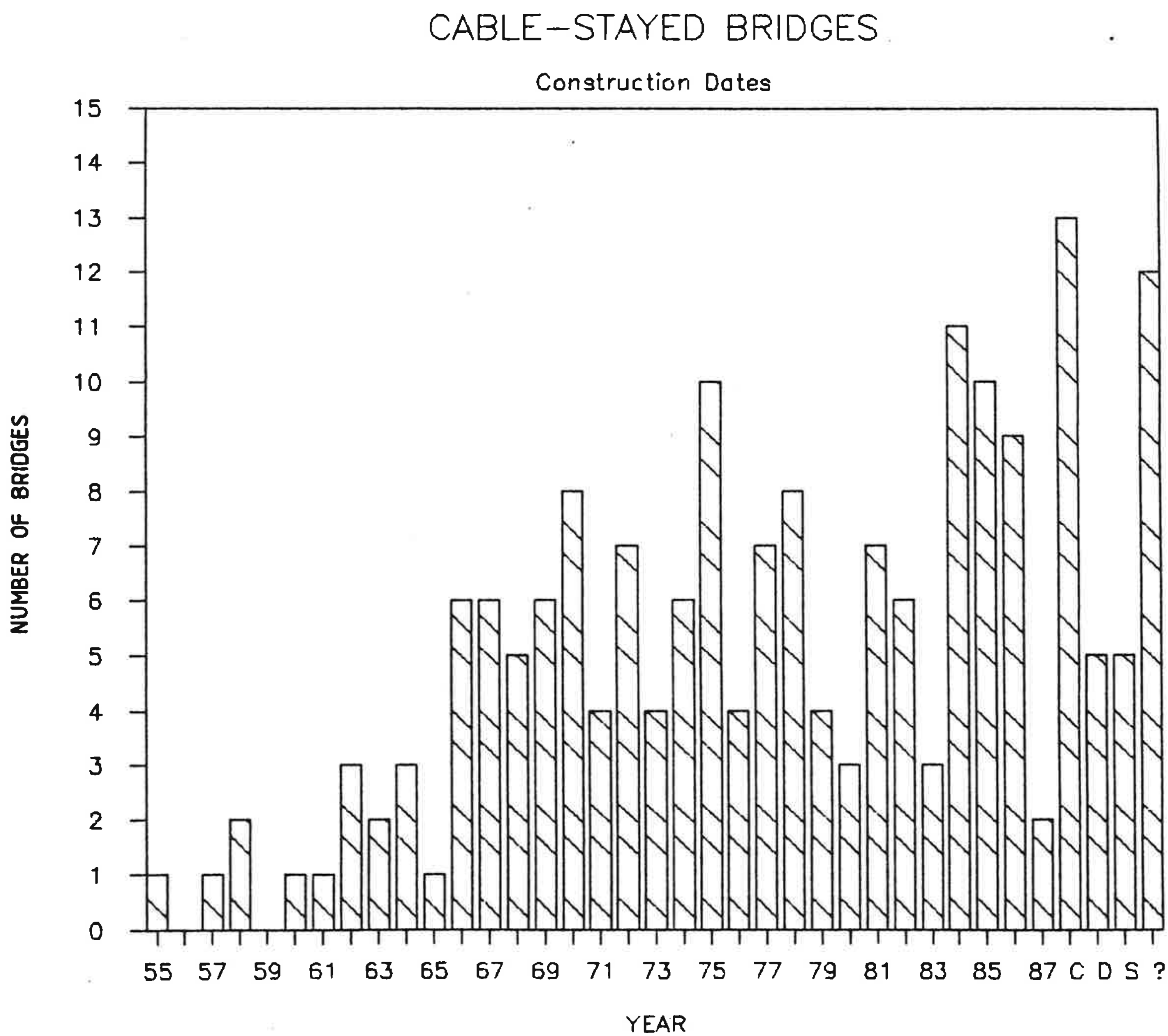
KEY:-

C = IN CONSTRUCTION

D = IN DESIGN

S = UNDER STUDY

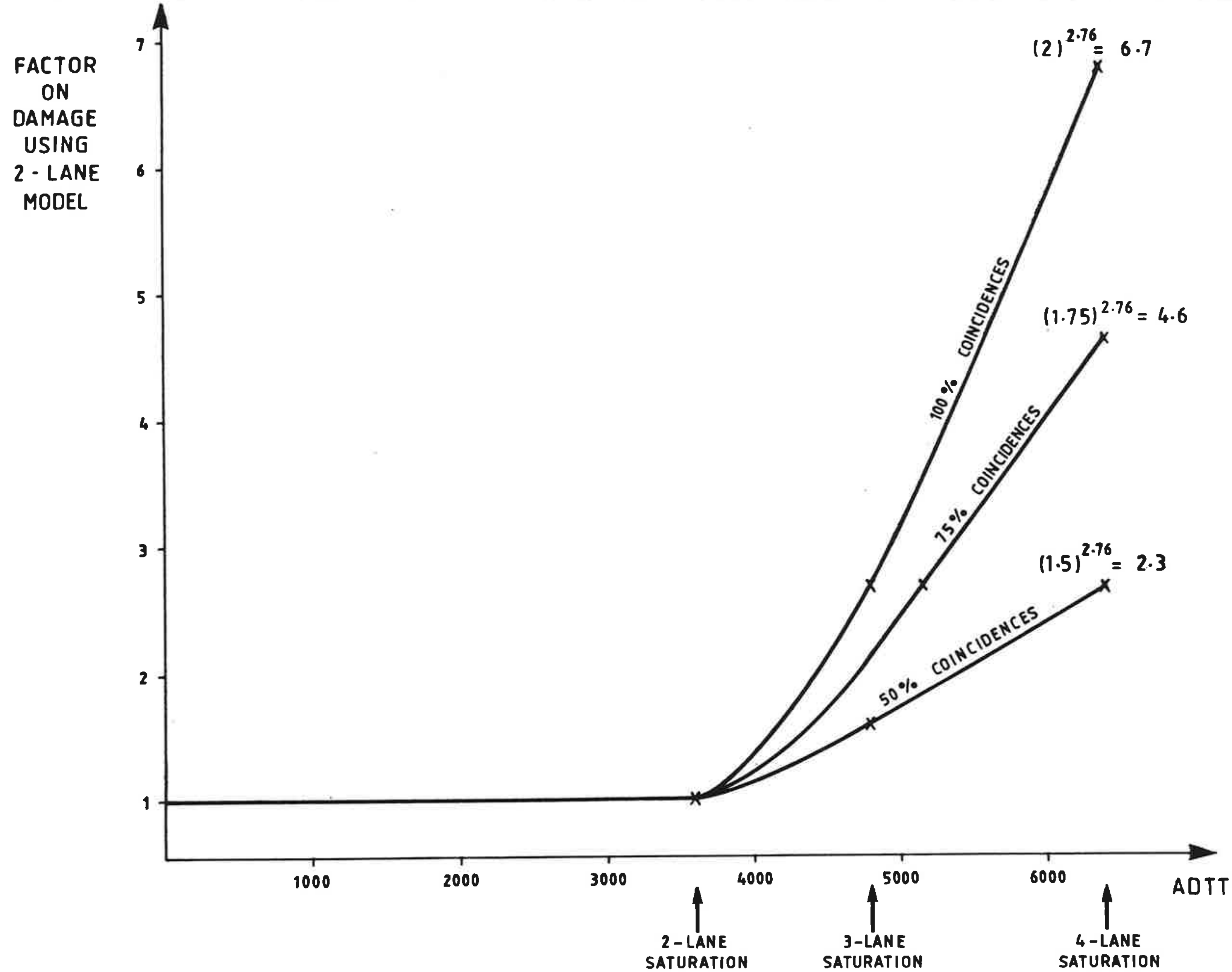
? = UNCERTAIN DATES



CABLE-STAYED BRIDGES - CONSTRUCTION DATES

FIG.10

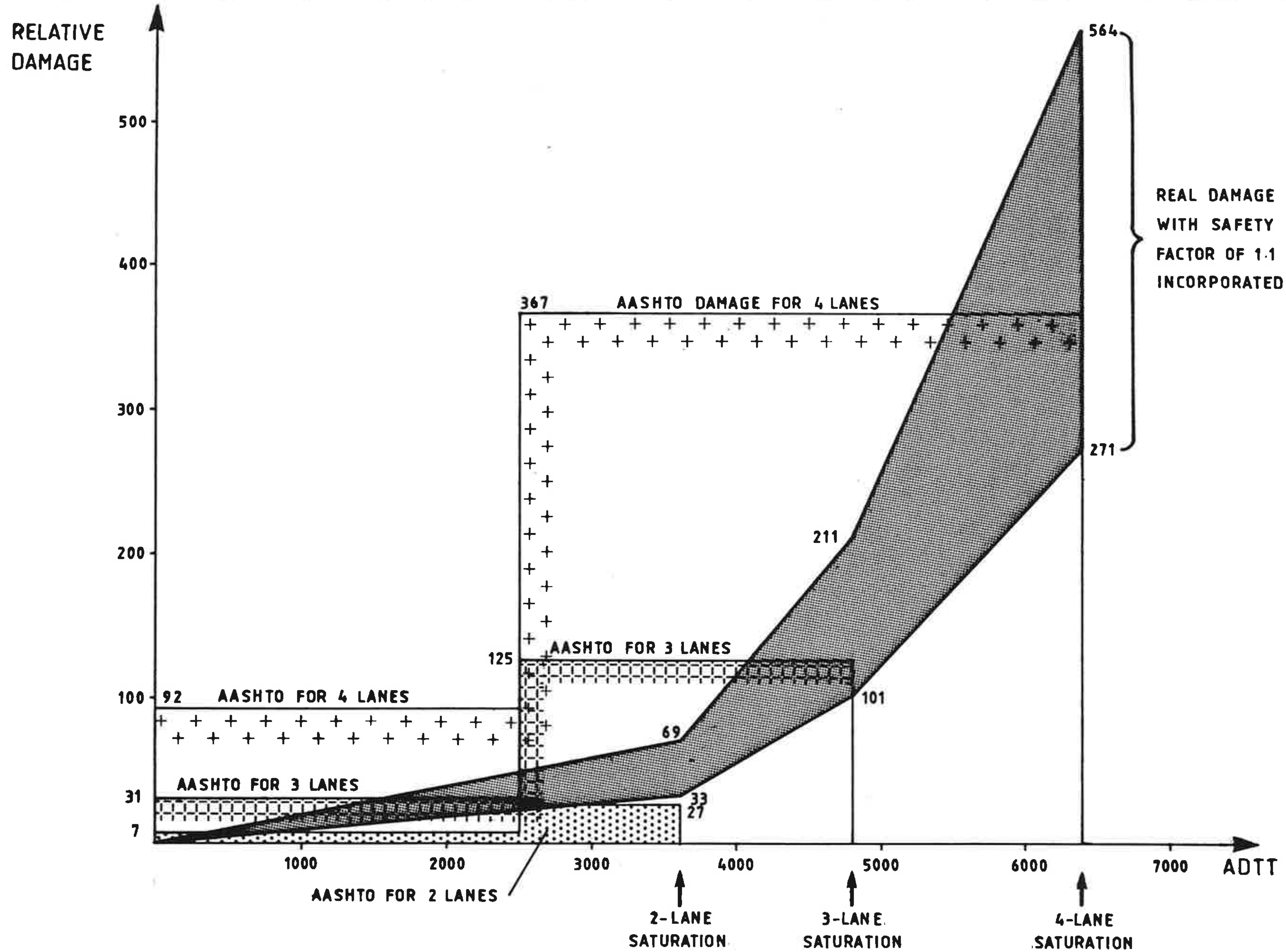
FIG.10



EFFECT OF INCREASING ADTT AND INCREASING NUMBER OF TRAFFIC LANES ON FATIGUE DAMAGE

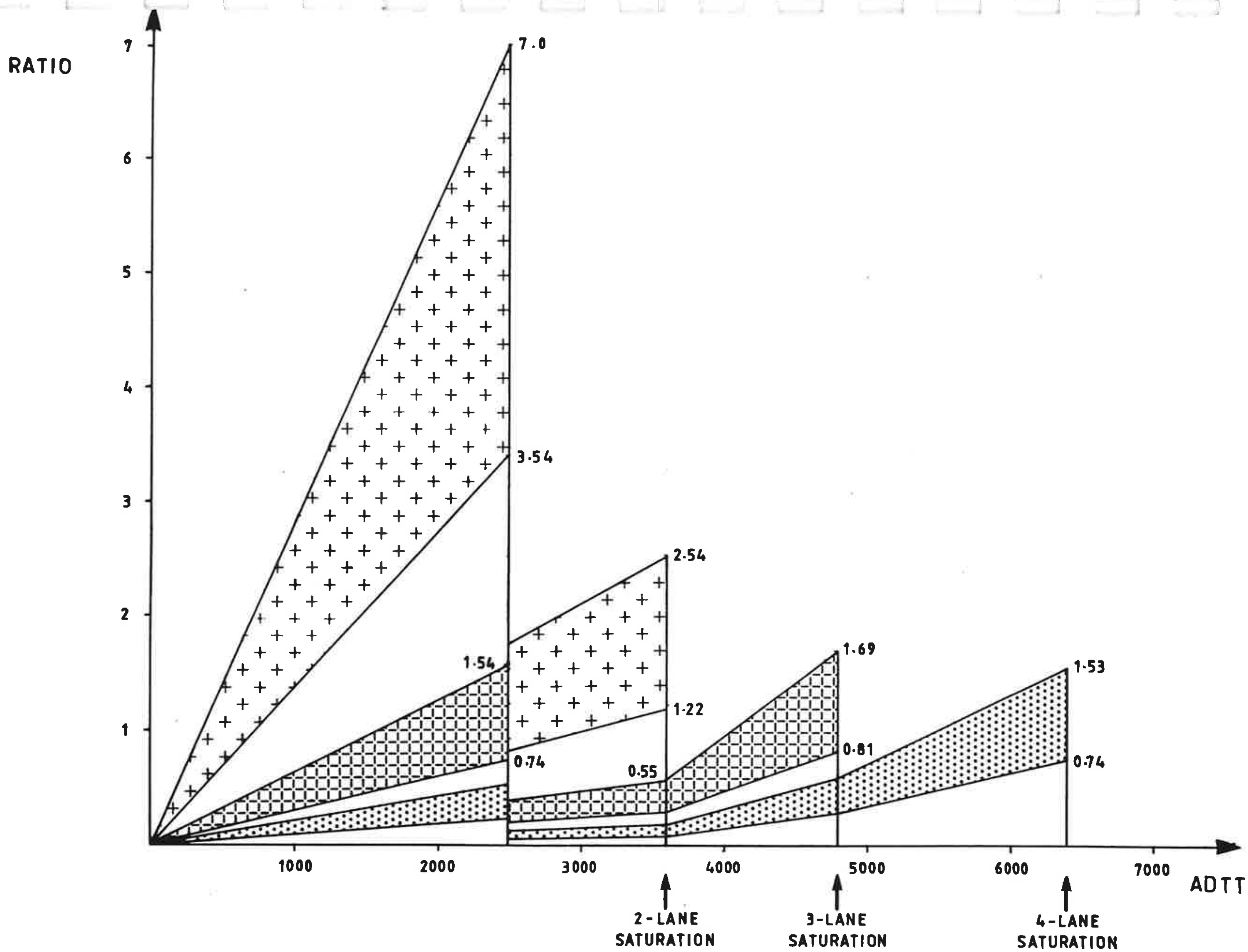
FIG. 11

FIG.11



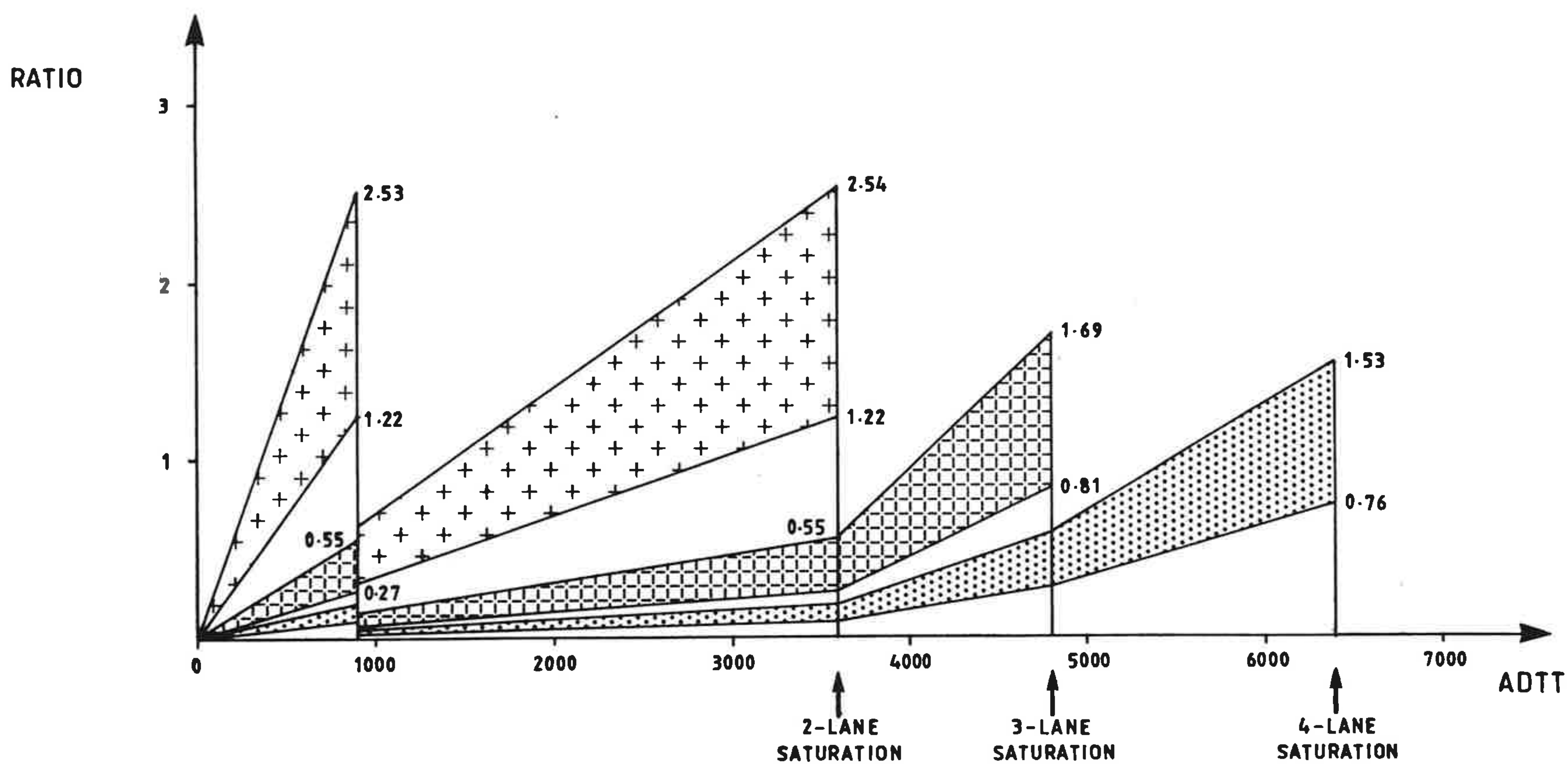
COMPARISON BETWEEN REAL AND CALCULATED DAMAGE EFFECTS

FIG. 12



RATIO OF REAL TO CALCULATED DAMAGE EFFECTS
(WITH CHANGE IN Nº OF CYCLES AT ADTT OF 2500)

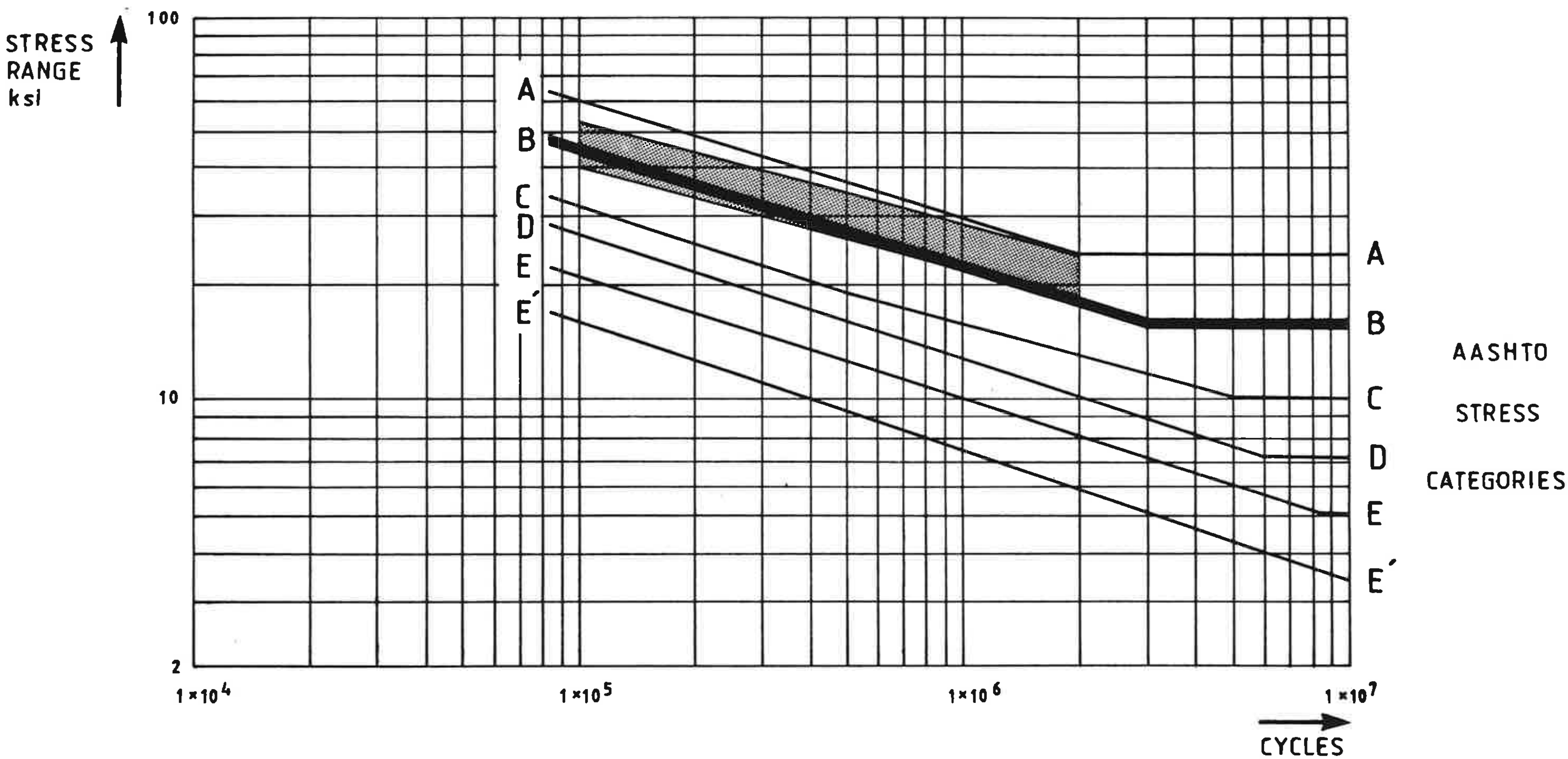
FIG. 13



RATIO OF REAL TO CALCULATED DAMAGE EFFECTS
(WITH CHANGE IN Nº OF CYCLES AT ADTT OF 900)

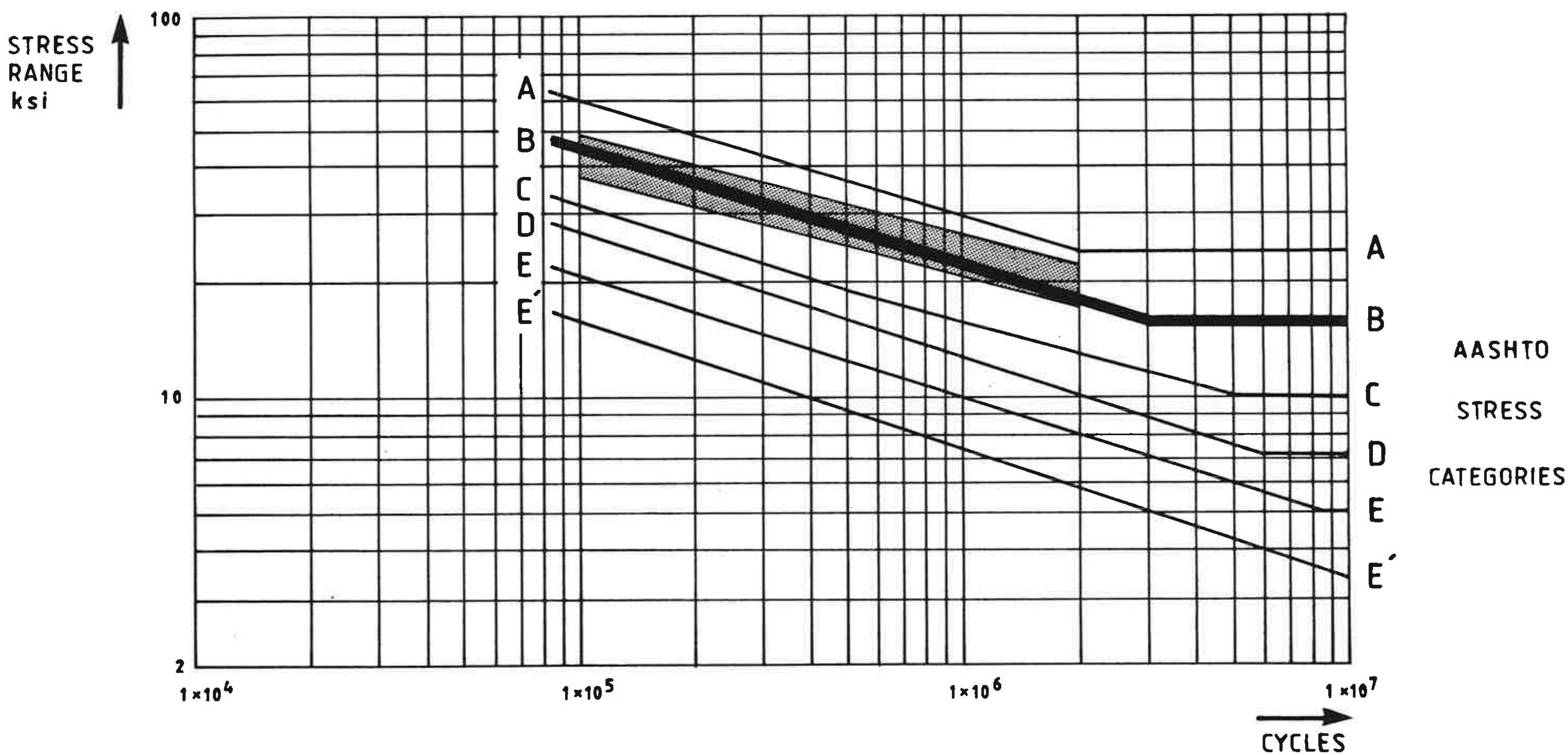
FIG. 14

FIG 14



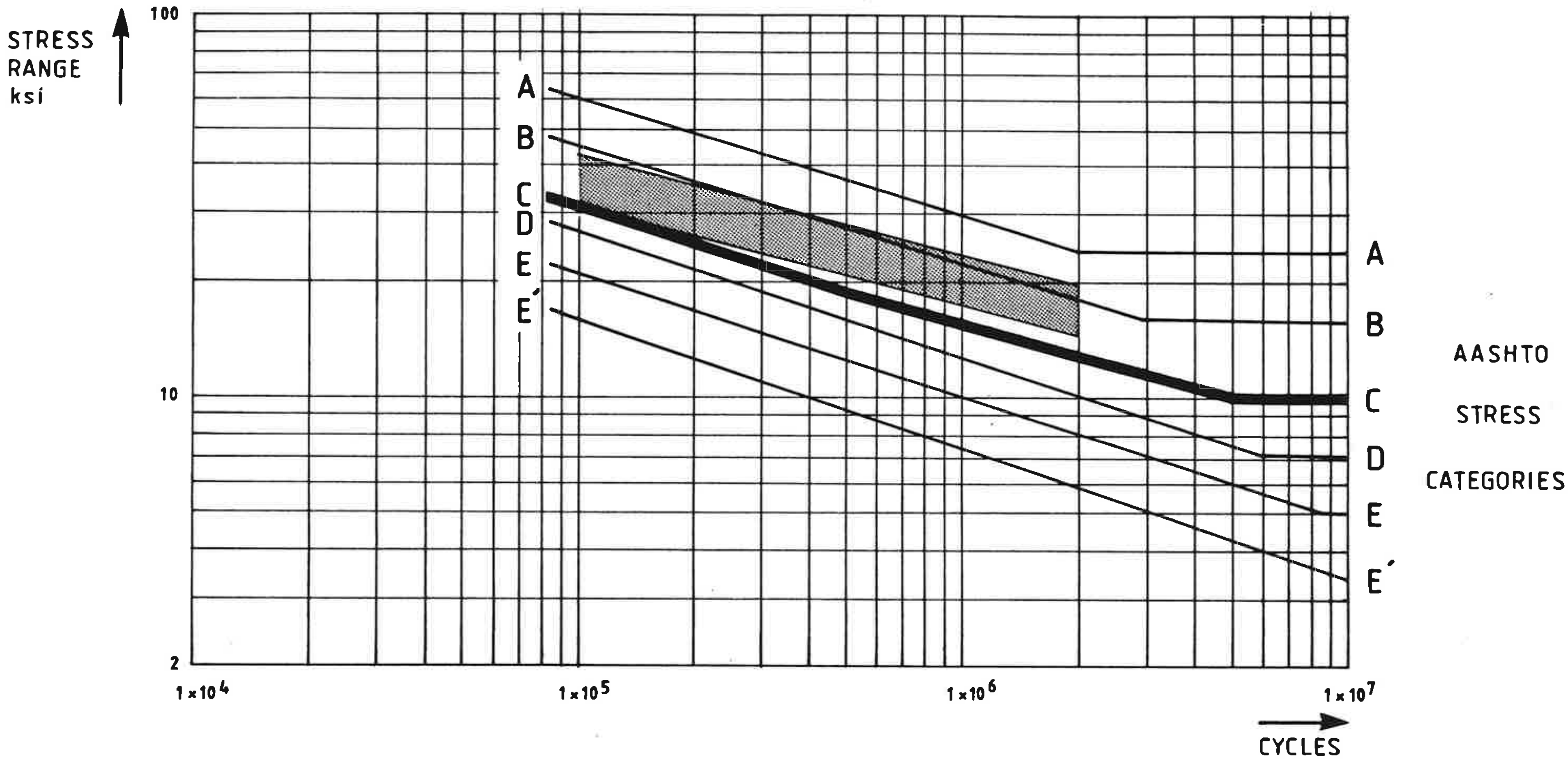
RECOMMENDED DESIGN STRESS RANGE FOR
7 WIRE STRAND, UNCOATED TO ASTM A416

FIG. 15



RECOMMENDED DESIGN STRESS RANGE FOR
UNCOATED WIRE TO ASTM A421

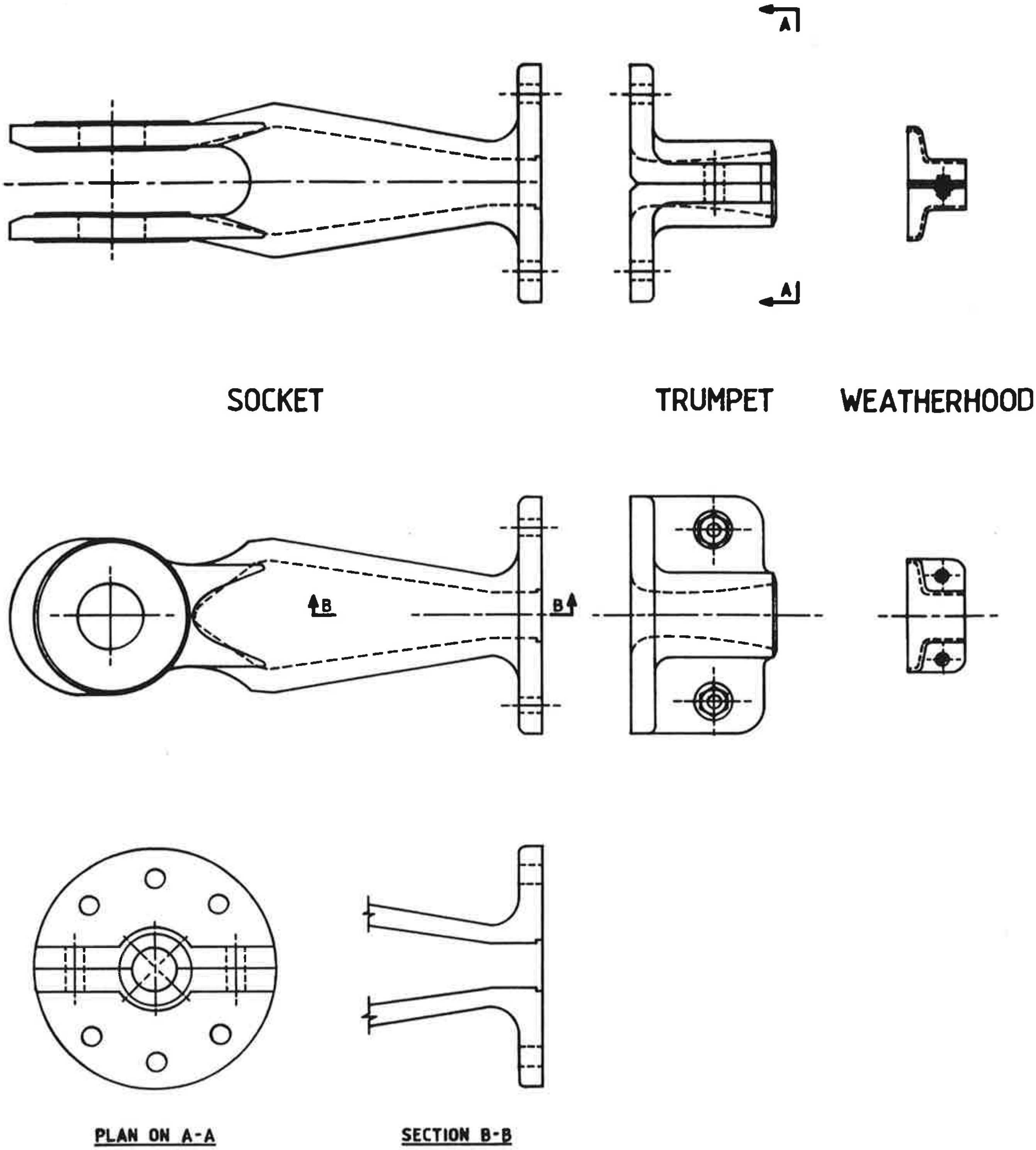
FIG. 16



RECOMMENDED DESIGN STRESS RANGE FOR
GALVANIZED BRIDGE STRAND TO ASTM A586

FIG. 17

FIG 17



TYPICAL ARRANGEMENT FOR SOCKET TRUMPET DETAIL

FIG. 18

APPENDIX A

APPENDIX A

CABLE-STAYED BRIDGES
DATABASE

This appendix contains a print out from the bridge database.

The bridges within each country are presented in descending order of main span length. The countries are presented in alphabetical order.

Where data is not available, a '-' at has been placed the relevant position in the database layout.

No. Name.   Date. Location.   Country.   Span. Side-Spans. Deck-S/C.
Cables.   Elevation.   Planes. Owner.   Designer.
Testing.   Questionaire-Reply. References.
Comments.

70 **MESOPOTAMIA**   1972 Parana   ARGENTINA   1115 417-417 Concrete
Locked Coil (?)   Single Stay   2   -   -
-   -   Ref 28,BBR(?)
-

13 **BRAZO LARGO**   1977 Rio Parana Guazo, Mesopotamia,B.A.   ARGENTINA   1083 361-361 Steel
Parallel wire PE grout   Radiating   2   -   Morandi,Leonhardt
-   -   Ref 28,179,BBR
Sometimes"Zarate":Rail+Highway: 337No7mm HiAm

146 **POSADAS ENCARNACION**   1986 Rio Parana (into PARAGUAY)   ARGENTINA   1083 - Concrete
Parallel wire PE grout   -   -   -   Consorcio Encarnacion
-   -   Ref 140,BBR
Rail+Highway : 325No7mm HiAm

66 **MANUEL BELGRANO (GENERAL)**   1973 Rio Parano, Chaco-Corrientes   ARGENTINA   804 433-433 Concrete
Locked Coil galvanised   Radiating   2   -   Ammann & Whitney (Ferrocemento +) Recchi
-   -   Ref 19,28,179
-

112 **WEST GATE**   1978 Lower Yarra, Melbourne, Victoria   AUSTRALIA   1100 470-470 Steel
Spiral Bridge Strand galv.   Fan   -   -   Freeman Fox & Ptnrs. (RDL)
-   -   Ref 28,179
-

07 **BATMAN**   1968 Launceston,Tasmania   AUSTRALIA   675 180 Steel
Locked Coil galvanised   Rad, Incl. Tower   2 inc.   -   G.Maunsell & Ptnrs. (Lewis,CBE)
-   Drawings   Ref 28,32,65,71
-

124 **BARWON RIVER FOOTBRIDGE**   1969 Geelong   AUSTRALIA   270 180-180 -
-   -   -   -   -
-   -   Ref 19
-

125 **MOUNT STREET FOOTBRIDGE**   1969 Perth   AUSTRALIA   117 117 -
Locked Coil galvanised   -   -   -   -
-   -   Ref 19
-

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

35 HAINSBURG 1973 River Danube AUSTRIA 748 453 Steel
Locked Coil Radiating 1(?) - Wagner Biro
- - Ref 28
-

61 LINZ 1972 Linz (Danube) AUSTRIA 705 236 Steel
Locked Coil Harp 1 - (Nobels-Peelman-Sotrahy)
- - Ref 28,167
-

101 STRALLATO 1975 Vienna (Danube Canal) AUSTRIA 390 183-183 Concrete
Parallel wire Single Stay 1 - -
- - Ref 19,35,36,Frey
Swung into final position

133 RAXSTRASSE FOOTBRIDGE - Vienna AUSTRIA ? - -
- - - - -
- - Ref 75
-

144 ALBERT CANAL 1985 Lanaye BELGIUM 761 - -
Parallel wire galv.PEtar epoxy- - - -
- - Ref 140,BBR
325No7mm HiAm

150 GODSHEIDE 1978 Hasselt BELGIUM 690 - Steel
- - - - Homberg
- - -
-

183 WANDRE Const.- BELGIUM 600 - Concrete
Parallel strand galv.individPE- - - -
- - Frey
-

182 SAMBRE Const.- BELGIUM 354 - Steel
Parallel strand galv.individPE- - - -
- - Frey
Rail bridge

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

122 CANAL DU CENTRE, FOOTBRIDGE 1966 Obourg BELGIUM 220 220 -
- - - - -
- - Ref 19
-

140 TILFF FOOTBRIDGE 1975 - BELGIUM 174 - -
Parallel wire PE grout - - - -
- - Ref 140,BBR
44No7mm BBRV

164 ALBERT CANAL 1983 Lixhe BELGIUM ? - -
Parallel wire galv.PEtar epoxy- - - -
- - BBR
208No7mm HiAm

184 BENHAIN - - BELGIUM ? - -
- - - - -
- - Frey
-

131 GERMAN PAVILLION FOOTBRIDGE 1958 Brussels BELGIUM ? - -
- - - - -
- - Ref 61
-

147 ZELZATE - - BELGIUM ? - -
Spiral Bridge Strand galv. - - - -
- - -
-

03 ANNACIS 1986 Vancouver, British Colombia (Fraser) CANADA 1526 600-600 Composite
Spiral Strand long lay galv.PEFan 2 - CBA-Buckland & Taylor (PLC,Paschen,Pike)
- YES . Ref 18,115,133,134,136
-

151 BURTON 1970 New Brunswick CANADA 850 - Steel
- - - - (Dominion Bridge)
- - -
-

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

82 **PAPINEAU-LEBLANC** 1969 Montreal CANADA 790 295-295 Steel
Spiral Bridge Strand galv. Radiating 1 Quebec Dept. of Highways Gendron,Lefebvre&Assoc (Dominion Bridge)
- - Ref 28,61,65,67,69,75
Cables sheathed in black polyethylene

37 **HAWKSHAW** 1967 Fredericton,New Brunswick CANADA 713 189-189 Steel
Spiral Bridge Strand Single Stay 2 New Brunswick Dept. of Trans. ADI Ltd.(Dominion Brigde)
- YES Ref 28,65
-

120 **LONGS CREEK** 1966 New Brunswick CANADA 713 190-190 -
Spiral Bridge Strand galv. - - - ADI Ltd.
- - Ref 8
-

161 **NACKAWIC RIVER** 1967 New Brunswick CANADA 216 54 -
- - - - Spear Northrup and Associates Ltd.
- - -
-

118 **GALIPEAULT** - Montreal CANADA ? - -
- - - - -
- - Ref 61
-

152 **YONGHE-TIANJIN** - Tianjin CHINA 853 - Concrete
- - - - -
- - -
-

48 **JINAN** 1981 - CHINA 722 150-150 Concrete
Parallel wire galv.Fab.in SituFan 2 - Soushen Shanshar Bingren Ronkun
- - Ref 115
-

126 **BARRANQUILLA** 1974 Barranquilla COLOMBIA 459 228-228 Concrete
- - - - Morandi (Cuellar Sovano & Gomez)
- - Ref 19
-

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

12 **BRATISLAVA** 1972 Danube CZECHOSLOVAKIA 994 246 Steel
Locked Coil Rad. Inclined Tower 2 inc. - A.Tesar (Vitkovice)
- - Ref 28,37,71,179
Called "Slovak National Uprising"

128 **FAROE** 1985 Storstrom Channel DENMARK 951 - Steel
Parallel wire PE grout - - - Cowinconsult (Monberg & Thorsen)
- - Ref 140,BBR
277No7mm HiAm

185 **ROVANIEMI** - - FINLAND ? - -
- - - - -
- - Frey
-

40 **HONFLEUR** Study Le Havre FRANCE 1670? - -
- - - - -
- - Ref 179
Study. Total length 2808(?) ft.

94 **ST.NAZAIRE** 1975 St.Nazaire (Loire) FRANCE 1325 518-518 Steel
Locked Coil Radiating 2 inc - (CFEM)
- - Ref 28,179,185,186
Last two layers of "Z" wires galvanised

14 **BROTONNE** 1977 Caudebec (Seine) FRANCE 1050 469-469 Concrete
Spiral Bridge Strand(?) Fan 1 - (Campenon Bernard Cetra)
- - Ref 18,19,28,32,37,62,104,179,187,Frey
-

67 **MASSENA** 1971 Paris (Seine) FRANCE 531 266-266 Steel
Locked Coil Harp 1 - (CFEM)
- - Ref 28
-

93 **ST.FLORENT** 1969 Le-Vieil (Loire) FRANCE 341 341 Steel
Locked Coil Radiating 2 - -
- - Ref 28,BCSA Conf.on Steel Bridges
-

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

179 MEYLAN FOOTBRIDGE 1979 - FRANCE 259 - Concrete
Parallel strand - - - -
- - Frey
-

58 LE HAVRE 1970 Le Havre FRANCE 240 105 Steel
Locked Coil Radiating 2 inc - -
- - Ref 28
-

180 ILLHOF FOOTBRIDGE 1980 - FRANCE 208 - Concrete
Parallel strand - - - -
- - Frey
-

31 ERSKINE 1971 Glasgow(Clyde), Scotland GREAT BRITAIN 1000 360-360 Steel
Spiral Bridge Strand galv. Single Stay 1 Scottish Development Dept. Freeman,Fox & Partners (Fairfield)
- YES Ref 28,32,42,66,179
-

52 KESSOCK 1982 Inverness, Scotland GREAT BRITAIN 787 262-262 Steel
Spiral Bridge Strand galv. Harp 2 Scottish Development Dept. Homberg (CBE,RDL)
- - Ref 32,147,148,149
-

113 WYE 1966 Nr.Chepstow GREAT BRITAIN 770 285-285 Steel
Spiral Bridge Strand galv. Single Stay 1 Dept. of Transport Freeman,Fox & Partners (CBE)
- YES Ref 28,32,106
-

34 GEORGE STREET 1964 Newport (Usk),Wales GREAT BRITAIN 500 56-56-56 Composite
Locked Coil galv. Harp + Splay 2 Gwent C.C. Mott Hay & Anderson (CBE)
- - Ref 28,32,61,66
-

64 LYNE M25 1982 Chertsey,London GREAT BRITAIN 180 180 P/S Conc.
Parallel wire PE grout Fan 2 Skew British Rail British Rail
- - Ref 19,32,159,BBR
Skew rail crossing 79No7mm DINA

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

72 MYTON BRIDGE (SWING) 1980 Kingston-upon-Hull GREAT BRITAIN 180 92 Steel
Spiral Bridge Strand Single Stay 1 Humberside C.C. Freeman,Fox & Partners (CBE)
- YES Ref 32
-

91 ROTHER VALLEY PARK FOOTBRIDGE 1985 Nr. Sheffield GREAT BRITAIN 115 72 Steel
Spiral Bridge Strand Radiating 2 inc Rother Valley Joint Committee South Yorkshire C.C.
- - Ref 145,146
-

86 PRESTON DOCKS (SWING) 1985 Preston GREAT BRITAIN 31 15.5M Steel
Parallel wire PE epoxy grout Single Stay 1 - L.G.Mouchel & Partners
- - Ref -
95No7mm HiAm

16 CHALKIS BRIDGE 1984 - GREECE 689 295-295 Concrete
- Radiating 2 - Schlaich & Partners
- - Ref 115
-

187 EVRIPOS - - GREECE ? - -
Parallel strand - - - Texeira Duarte Huarte
- - Frey
-

117 COTTON TREE DRIVE FOOTBRIDGE 1979 Cotton Tree Drive HONG KONG 162 ?110 Concrete
Parallel strand Radiating 2 Architectural Services Department Freeman,Fox & Partners
- YES Ref FFP(F.E.),Frey
-

01 AIRPORT HOTEL FOOTBRIDGE 1982 Kai Tak Airport HONG KONG 156 124 Concrete
Parallel strand Poly-Tube VSL Radiating 1 Regal Meridian Airport Hotel Freeman Fox (Far East)
- YES,+Paper by TaylorRef 18
-

41 HOOGHLY (Second) Const Calcutta INDIA 1499 600-600 Composite
Parallel wire PE grout Radiating 2 - Leonhardt & Andra, Freeman Fox
- - Ref 28,140,179,BBR
Constructing: 227No7mm HiAm

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

153 ADHAMIVAH 1984 Baghdad IRAQ 599 - Steel
- - - - G.Maunsell, (Marubeni)
- - -
-

85 POLCEVERA 1967 Genoa ITALY 689 663-459 Concrete
Locked Coil Single Stay 2 - Morandi (SICAR)
- - Ref 6,19,28
-

141 ARNO 1977 Florence ITALY 676 - Steel
Parallel wire PE grout - - - De Miranda (CMF)
- - Ref 140,BBR
365No7mm HiAm

127 CARPINETO 1977 Poetenza Province ITALY 594 100-100 Concrete
- - - - -
- - Ref 19
-

04 ANSA DE MAGLIANA 1967 Rome ITALY 476 177 Concrete
Locked Coil Single Stay 2 - Morandi
- - Ref 6,19,28
-

107 TIBER BRIDGE 1981 Rome ITALY 354 118-118 Steel
Parallel strand Fan 1 City of Rome Ing.E. MENCARELLI, Rome
- - Ref 18
Contractor Ing.A.Lollini SPA : VSL

115 YOKOHAMA BAY Const Tokyo JAPAN 1509 - Steel
- - 2 - Shin Nihon Giken (IHI +)
- - Ref 40
-

38 HITSUISHI JIMA Const Kurashiki, Shikoku JAPAN 1378 607-607 Steel
Parallel wire galv.PE PBPinj. Fan 2 - Tokyo Eng (IHI +)
- - Ref 37,132,157,BBR
Constructing: 277No7mm HiAm

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

134 IWAKURO JIMA Const Okayama, Shikoku JAPAN 1378 607-607 Steel
Parallel wire galv.PE PBPinj. Fan 2 - -
- - Ref 37,132,157,BBR
Constructing: 277No7mm HiAm

145 MEIKO-NISHI 1985 Nagoya JAPAN 1329 - Steel
Parallel wire galv+polyPEgrout- - - IHI Consortium
- - Ref 140,BBR
379No5mm HiAm

130 YAMATOGAWA 1982 Osaka JAPAN 1165 - Steel
Parallel wire - - - Mitsubishi
- - Ref 39
Plastic sheath

155 AJIGAWA Const.Osaka JAPAN 1148 - Steel
Parallel wire PE grout - - - Sogo Gijuts (IHI +)
- - BBR
349No7mm HiAm

46 ISHIKARI 1972 Hokaido JAPAN 820 361-361 Steel
Parallel wire Fan 2 inc - -
- - Ref 28
Plastic sheath

103 SUEHIRO 1975 Tokushima Prefecture JAPAN 820 361-361 Steel
Parallel wire Fan 1 - Chodai (Nipon Kokan-Kawasaki)
- - Ref 28,41
Plastic sheath

51 KAMONE 1975 Osaka JAPAN 787 328-328 Steel
Parallel wire Fan 1 - -
- - Ref 28
Plastic sheath

89 ROKKO 1977 Kobe JAPAN 722 - Steel
Parallel wire Fan 2 - (Dominion Bridge)
- - Ref -
-

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

154 TOYOSATO 1969 Osaka JAPAN 709 - Steel
Parallel wire - - - (Mitsubishi-Hitachi-Fuji-Yawata)
- - -
Plastic sheath

81 ONOMICHI 1970 - JAPAN 705 279-279 Steel
Locked Coil Radiating 2 - -
- - Ref 28,71,74
-

114 YODO 1970 Osaka JAPAN 656 407 Steel
Parallel wire Fan 1 - -
- - Ref 2,28,61
-

22 DIAKAKA 1975 Yokohama JAPAN 541 328 Steel
Parallel wire Radiating 2 - -
- - Ref 28
Plastic sheath

05 ARAKAWA 1970 Tokyo JAPAN 525 197-197 Steel
Parallel wire Harp 1 - -
- - Ref 28
Plastic sheath. Also called "Komatsugawa"?

69 MAYA 1966 Kobe Harbour JAPAN 459 226 Steel
Locked Coil Fan 1 - -
- - Ref 28
-

43 HORIKOSHI BRIDGE 1984 Hachioji City JAPAN 220 122 Concrete
- Fan 2 HORIKOSHI Private school, HACHIOJI CITY -
- - Ref 18
-

181 NISHIGAOKA 1982 - JAPAN 171 - Concrete
Parallel strand - - - -
- - Frey
-

No. Name.   Date. Location.   Country.   Span. Side-Spans. Deck-S/C.
Cables.   Elevation.   Planes. Owner.   Designer.
Testing.   Questionaire-Reply. References.
Comments.

167 BOTAN FOOTBRIDGE   1982 Saitama Prefecture   JAPAN   ? -   -
Parallel wire PE grout   -   -   -   -
-   -   BBR
91No7mm HiAm

174 CHICHIBU   1984 Chichibu, Saitama Prefecture   JAPAN   ? -   -
Parallel wire PE grout   -   -   -   -
-   -   BBR
409No7mm HiAm

163 FUJITO & HINOURA   1980 Aioi   JAPAN   ? -   -
Parallel wire PE grout   -   -   -   -
-   -   BBR
121No7mm HiAm

162 KAWAHARA FOOTBRIDGE   1979 Morioka   JAPAN   ? -   -
Parallel wire PE grout   -   -   -   -
-   -   BBR
37No7mm HiAm

171 SHIN KAMIHIRAI   1986 Tokyo   JAPAN   ? -   -
Parallel wire PE grout   -   -   -   -
-   -   BBR
313No7mm HiAm

173 SHIN TAKANASHI   1984 Gozu City, Shimane Prefecture   JAPAN   ? -   -
Parallel strand PE alumin.case-   -   -   -
-   -   BBR
19No15mm-2" strands HiAm

97 SHINNO   1963 -   JAPAN   ? -   -
-   -   2   -   -
-   -   Ref 35(Pg.47)
-

175 SUZAKI NISHI KOEN   1985 Fukuoka   JAPAN   ? -   -
Parallel wire PE grout   -   -   -   -
-   -   BBR
241No7mm HiAm

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

178 TSUKUHARA 1986 Donto-Dam JAPAN ? - -
Parallel wire extrudePE grease- - - -
- - BBR
15No0.6" HiAm

172 YASAKA 1986 Otake City JAPAN ? - -
Parallel wire PE grout - - - -
- - BBR
301No7mm HiAm

166 YASURAGI FOOTBRIDGE 1981 Fukumitsu JAPAN ? - -
Parallel wire PE grout - - - -
- - BBR
55No7mm HiAm

176 YODOGAWA 1986 Osaka JAPAN ? - -
Parallel wire PE polymer grout- - - -
- - BBR
421No7mm HiAm

49 JINDO 1984 - KOREA 1129 230-230 Steel
Locked Coil Fan 2 inc IRI Reg.Const.Off.Min.Const,Rep.Korea Rendel Palmer & Tritton
- YES Ref 143,144
Drgs. with RPT

26 DOLSAN 1984 - KOREA 919 279-279 Steel
Locked Coil Fan 2 inc. IRI Reg.Const.Off.Min.Const,Rep.Korea Rendel Palmer & Tritton
- YES Ref 143,144
Drgs. with RPT

80 OLYMPIC BRIDGE 1984 - KOREA 492 492 -
- Fan 1 - Samwoo Engineering Co.LTD.
- - Ref 115
-

110 WADI KUFF GORGE 1971 Sipac, Beida LIBYA 925 312-312 Concrete
Locked Coil Single Stay 2 - Morandi (Cogefar)
- - Ref 6,19,28,61,179
-

No. Name. Cables. Testing. Comments.	Date. Location. Elevation. Planes. Owner. Questionaire-Reply. References.	Country.	Span. Side-Spans. Deck-S/C. Designer.
138 DIEKIRCH Parallel wire - -	1974 - - - - - Ref 140	LUXEMBURG	164 - - -
84 PENANG BRIDGE Parallel Bars - -	1985 Penang Island Harp 2 - YES Ref 115	MALAYSIA	738 353-353 Concrete H N T B (Hyundai)
106 TAMPICO Parallel strand - -	1986 Panuco River Harp 1 - - Ref 115,BBR	MEXICO	1180 230-230 Steel Comec-Sogelerg (ICA Group)
19 COATZACOALCOS Parallel strand - -	1984 Coatzacoalcos River Fan 1 - - Ref 17,115,Frey	MEXICO	945 565-531 Concrete Comec-Sogelerg (ICA Group)
158 EWIJK - - -	1975 Waal River - - - - -	NETHERLANDS	919 - Steel -
157 WILLEMS - - -	1981 Roterdam ? - - - - -	NETHERLANDS	886 - Steel -
108 TIEL Locked Coil galv. - -	1974 Tiel (Waal) Radiating 2 - - Ref 19,28,179	NETHERLANDS	876 312-312 Concrete Van Hattum en Blankenvoart
142 IJSSEL Parallel wire PE grout - 149No7mm HiAm	1983 Kampen - - - - Ref 140,BBR	NETHERLANDS	635 - - -

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

132 GALECOP 1971 Utrecht NETHERLANDS 590 - Steel
Locked Coil galv. - - - Rijkswaterstaat
- - Ref 75
-

36 HARMSEN 1968 Brielle NETHERLANDS 354 154 Steel
Locked Coil Harp + Star 1 - -
- - Ref 28
-

100 STAVANGER BRIDGE 1976 - NORWAY 607 144 Conc+Steel
- Radiating 2 inc - -
- - Ref 28
-

149 YANANGO - - PERU 489 - -
Spiral Bridge Strand galv. - - - -
- - -
-

123 PRETORIA PIPE BRIDGE 1968 Pretoria SOUTH AFRICA 93 93 -
- - - - -
- - Ref 19
-

06 BARRIOS DE LUNA 1983 Cordillera SPAIN 1444 324-324 Concrete
Parallel strand Fan 2 - Fernandez Casado (Huarte Y CIA)
- - Ref 115,176,Frey
-

109 VIGO (PUENTE DE RANDE) 1978 Vigo SPAIN 1313 484-484 Steel
Parallel strand Fan 2 Autopistas del Atlantico -
- - Ref 37,179,Frey
-

30 EBRO RIVER 1978 Navarra SPAIN 480 - Concrete
Locked Coil Fan Centre Tower 1-2 inc - Fernandez Casado
- - Ref 115
-

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

02 ALCOY BRIDGE (FERNANDO REIG) 1986 - SPAIN 433 354 Concrete
Parallel strand Fan 2 - Ordonez Calzon Mato Bonet Ventura(Dragos
- - Ref 115, NCE 7/8/86
-

65 M-25 FOOTBRIDGE 1967 - SPAIN 281 69-69 Concrete
Parallel wire Fan + Star 2 - Fernandez Casado
- - Ref 115
-

121 TEMPUL AQUEDUCT 1925 Guadalete SPAIN 198 66-66 Concrete
- - - - Fernandez Casado
- - Ref 19
-

148 NERVION RIVER - Bilbao SPAIN ? - -
Locked Coil galv. - - - -
- - -
-

186 GUADIANA - - SPAIN / PORTUGAL 1148 ?- -
- - - - -
- - Frey
-

156 TJORN 1981 Asker Fjord SWEDEN 1201 - Steel
- - - - Skanska Cementgjuteriet-Krupp
- - -
-

102 STROMSUND 1955 Stromsund SWEDEN 600 243-243 Steel
Locked Coil Radiating 2 - Demag
- - Ref 18,28,61,66,75
-

143 DIEPOLDSAU 1985 Rhine SWITZERLAND 318 - -
Parallel wire PE grout - - - -
- - Ref 140,BBR
77No7mm DINA

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

136 REUSS 1970 Bremgarten SWITZERLAND 217 - -
Parallel wire - - - -
- - Ref 140
-

135 BIRS FOOTBRIDGE 1962 Basel SWITZERLAND 164 - -
Parallel wire PE grout - - - -
- - Ref 140,BBR
40No6mm BBRV

60 LIEBRUTI FOOTBRIDGE 1978 Kaiseraugst SWITZERLAND 153 58 Concrete
Parallel strand Radiating 1 Persian & Social Fund of Hoffman EMCH & BERGER AG Berne
- - Ref 18
VSL

169 N1 FOOTBRIDGE 1976 Wallisellen SWITZERLAND ? - -
Parallel wire PE grout - - - -
- - BBR
55No7mm BBRV

177 ST-MAURICE 1986 Rhone, St-Maurice SWITZERLAND ? - -
Parallel wire PE grout - - - -
- - BBR
298No7mm DINA & BBRV

170 TSCHACHEN RIVER 1973 Schwanden SWITZERLAND ? - -
Parallel wire PE grout - - - -
- - BBR
105No7mm BBRV

18 CHUNG YANG 1984 Taipei TAIWAN 656 303-303 Steel
Parallel strand Fan 2 Tai.Prov.Govern. China Eng.Inc.(Tang Eng.IronWorks)
- Drgs. Ref 115
-

57 KWANG FU 1978 Taipei TAIWAN 440 220, 4 spansConcrete
Parallel strand Radiating 2 Taiwan Highway Bureau Churgsin Engineering Co.Ltd., T.Y.Lin
- Drgs. Ref 19,36,Frey
-

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

165 KIRUMI 1985 Mara River TANZANIA ? - -
Parallel wire PE grout - - - -
- - BBR
151No7mm HiAm

17 CHAO PHYA Const Bangkok THAILAND 1476 201-201-201 Steel
Locked Coil Fan 1 - Peter Fraenkel, Homberg
- YES Ref 127,154
Sockets loosened,twisted on stay

23 DAME POINT Const Jacksonville (St.Johns) Florida USA 1300 650-650 Concrete
Parallel Bars threaded Harp 2 Jacksonville Transportation Authority H N T B (Pensaccia-Tyger)
- YES Ref 19,28,115,179,180
Const. Complete'88

08 BAYTOWN Design Houston Ship Channel, Texas USA 1250 413-413 Steel
Parallel strand Radiating or Fan 2 - Greiner
- YES Ref 124,180
Designing

63 LULING 1984 New Orleans, Louisiana USA 1222 508-495 Steel
Parallel wire PE grout Radiating 2 Louisiana DoTD Frankland & Lienhard, Modjeski & Masters
- YES Ref 10,12,28,119,131,140,179,180,BBR,Fry
307No.25in HiAm

104 SUNSHINE SKYWAY 1987 Tampa Bay, Florida USA 1200 540-540 Concrete
Parallel strands Fan 1 State of Florida Dept. of Transportation Figg & Muller Inc. (Paschen)
Swiss Fed.Labs.Dubendorf YES Ref 18,115,135,152,180
-

90 ROOSEVELT LAKE Design Roosevelt, Arizona USA 1100 - Concrete
- - - Arizona Department of Transportation Figg&Muller(Conc) HNTB(Steel)
- YES Ref 180
Designing Let Nov.86 Complete 89?

105 TALMADGE, MEMORIAL Design Savannah, Georgia USA 1100 470-470 S/C
- Fan 2 Georgia D.O.T. Greiner,DRC(M.ChungTang)+Parsons..
- YES Ref 180
Designing Let 86-87 Constr.4yrs

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

83 PASCO - KENNEWICK 1978 Colombia, Washington State USA 980 407-407 Concrete
Parallel wire PE grout Radiating 2 City of Kennewick Arvid Grant (Kiewit)
- YES Ref 10,18,19,28,32,102,103,161,179,180,+
Also Ref 195,BBR,Frey 283No.25in HiAm

45 HUNTINGTON EAST 1985 Ohio River, West Virginia USA 900 450 Concrete
Parallel wire PE grout Radiating 2 inc W.Virginia Dept.of Highways Arvid Grant (Melbourne Bros.)
- YES Ref 28,140,179,180,BBR
307No.25in HiAm

87 QUINCY Const Quincy, Illinois USA 900 440-440 Composite
Parallel strand epoxy PE groutFan 2 Illinois D.T.O. Modjeski & Masters (McCarthy Bros.)
- YES Ref 115,180
Constructing Complete Fall 1987

111 WEIRTON - STEUBENVILLE 1987 West Virginia USA 820 690 Composite
Parallel strand Fan 2 inc W.Virginia Dept.of Highways Michael Baker Jr. INC. (Groves)
- YES Ref 28,115,180
-

20 COCHRANE Const Mobile,Alabama USA 800 360-360 Concrete
Parallel strand Fan 2 Alabama State Highway Dept. Barret, Daffin & Carlin +Iberenza
- YES Ref 115,180
-

15 BURLINGTON Design Burlington, Iowa USA 660 405 S/C
- Fan 2 Iowa D.O.T. Svedrup & Parcel
- YES Ref 180
Designing Complete 1991

73 NECHES RIVER Const Port Arthur, Texas USA 640 280-280 Concrete
Parallel strand + Saddle Harp 2 - Figg & Muller
- YES Ref 124,180
Complete 1990

47 JAMES RIVER Const Richmond, Virginia USA 630 150-150-150 Concrete
Parallel strand Harp 1 Virginia Dept.of Highways&Transport. Figg & Muller, (Groves, Kiewit)
- YES Ref 115,180
Constructing Complete June 1988. Stronghold

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

98 SITKA 1972 Sitka,Alaska USA 450 151-151 Composite
Spiral Bridge Strand Single Stay 2 Alaska DOT State of Alaska
- YES Ref 28,68,75,150,180
-

71 MOORE (CAPT. WILLIAM) 1976 Near Skagway,Alaska USA 250 30 Steel+Timb
Spiral Bridge Strand Galv. Single Stay 2 Alaska DOT State of Alaska
- YES Ref 180
-

119 MENOMONEE FALLS FOOTBRIDGE 1970 Wisconsin USA 217 72-72 Composite
Spiral Bridge Strand galv. Single Stay 2 Incl. Wisconsin DOT Wisconsin DOT
- YES Ref 64
3" dia.

92 SACRAMENTO RIVER (SWING) 1977 Meridian, California USA 180 180 Composite
Spiral Bridge Strand(?) Radiating 2 - State of California
- Drgs. Ref 180
-

09 BENTON CITY, YAKIMA 1957 Kiona, Washington USA 170 54-54 Steel+Conc
Fabricated Steel Section Single Stay 2 Benton County, Washington Homer M.Hadley
- YES Ref 19,35
-

10 BLAIR Study Iowa USA ? - -
- - - - -
- - Ref 180
-

116 CLARK BRIDGE Study Illinois USA ? - -
- - - Illinois D.T.O. -
- YES Ref -
Study

21 COOPER RIVER Design Charleston, South Carolina USA ? - -
- - - South Carolina D.O.T. Figg & Muller
- YES Ref 180
Cable Stayed not prefered

No.	Name.	Date.	Location.	Country.	Span.	Side-Spans.	Deck-S/C.	Cables.	Elevation.	Planes.	Owner.	Designer.	Testing.	Questionaire-Reply.	References.	Comments.
42	HOOVER DAM	Study	Colorado	USA	?	-	-	-	-	-	-	-	-	-	Ref -	Study
53	KNICK ARM	Study	Alaska	USA	?	-	-	-	-	-	-	-	-	-	Ref 180	Study
159	DAUGARA RIVER	-	Riga	USSR	1024	-	Steel	-	-	-	-	-	-	-	-	-
160	DNIEPER RIVER	1976	Kiev, Ukraine	USSR	984	-	Steel	-	-	-	-	Sojuzdorproject Institute, (Mostostroj)	-	-	-	-
25	DNIEPER	1964	Kiev	USSR	472	217-217	Concrete	-	Radiating	2	-	-	-	-	Ref 19,28	-
33	GENERAL RAFAEL URDANETA	1962	Lake Maracaibo	VENEZUELA	770	5 Spans	Conc.Prest	Locked Coil	Single Stay	2	-	Morandi	-	-	Ref 3,6,19,28,32,61,71	-
129	ANGOSTURITA	-	-	VENEZUELA	?	-	-	-	-	-	-	-	-	-	Ref 37	D E L E T E
29	DUSSELDORF - FLEHE	1979	Dusseldorf (Rhine)	WEST GERMANY	1205	-	Steel	-	-	-	-	Leonhardt & Andra	-	-	Ref 37,179	-

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

27 DUISBURG-NEUENKAMP   1970 Duisburg (Rhine)   WEST GERMANY   1148 270-153 Steel
Locked Coil   Fan   1   -   -
-    -   Ref 28,71,179
-

55 KOHLBRAND   1974 Hamburg   WEST GERMANY   1066 320-320 Steel
Locked Coil   Fan   2 inc   -   Rheinstahl-Lehmann
-    -   Ref 28,32,179
Recabled,tower jacked.New, PE epoxy? Thyssen

54 KNIEBRUCKE   1969 Dusseldorf (Rhine)   WEST GERMANY   1050 160-160-160 Steel
Locked Coil   Harp   2   -   Leonhardt (Demag Lehmann)
-    -   Ref 8,28,37,63,65,66,71,179,194
-

96 SEVERINS BRUCKE   1960 Cologne (Rhine)   WEST GERMANY   991 495 Steel
Locked Coil   Radiating   2 inc   -   -
-    -   Ref 28,61,65,66,71,75,179,194
-

74 NEUWIED   1977 (Rhine)   WEST GERMANY   958 771 Steel
Locked Coil   Radiating   1   -   Homberg, Leonhardt, (CFEM)
-    -   Ref 28,66
-

24 DEGGENAU   1975 Deggendorf, (Danube)   WEST GERMANY   951 476 Steel
Locked Coil   Radiating   1   Autobahndirektion Sudbayern, Munchen   Design & Construct M.A.N. AG.
-    YES   Ref 28,179
-

56 KURT SCHUMACHER   1972 Mannheim (Rhine)   WEST GERMANY   942 205 Steel
Parallel wire polymethan+tape Radiating   2 inc   -   Leonhardt & Andra, (Lehmann)
-    -   Ref 28,37,66,179,BBR
Often "Mannheim" Road+rail   HiAm anchorage

11 BONN-NORD (FRIEDRICH-EBERT)   1966 Bonn   WEST GERMANY   919 394-394 Steel
Locked Coil   Fan   1   -   (Hein Lehmann)
-    -   Ref 28,61,65,66,71
-

No. Name.   Date. Location.   Country.   Span. Side-Spans. Deck-S/C.
Cables.   Elevation.   Planes. Owner.   Designer.
Testing.   Questionaire-Reply. References.
Comments.

59 **LEVERKUSEN**   1964 Leverkusen   WEST GERMANY   919 348-348 Steel
Locked Coil   Harp   1   -   Homberg, (Kloenne)
-    -   Ref 28,65
-

99 **SPEYER**   1975 Mannheim (Rhine)   WEST GERMANY   902 198 Steel
Locked Coil   Radiating (Half)   2 inc   -   (Kloenne)
-    -   Ref 28,179
Sockets loosened,twisted on stay

76 **NORTH BRIDGE (THEODOR-HEUSS)**   1958 Dusseldorf   WEST GERMANY   853 354-354 Steel
Locked Coil   Harp   2   -   Leonhardt, (Demag, Lehmann)
-    -   Ref 28,61,66,71,194
-

79 **OBERKASSEL**   1974 Dusseldorf (Rhine)   WEST GERMANY   846 212 Steel
Locked Coil   Harp   1   -   Leonhardt, (Lehmann)
-    -   Ref 28,37,66,71,179,194
-

28 **DUISBURG-RHEINHAUSEN**   1965 (Rhine)   WEST GERMANY   837 - -
-    -   -   -   -
-    -   Ref 179
-

88 **REES - KOHLUR**   1967 Rees (Rhine)   WEST GERMANY   837 341-341 Steel
Locked Coil   Harp   2   -   Homberg, (Demag)
-    -   Ref 28,66,71,179
-

68 **MAXAU**   1966 Karlsruhe   WEST GERMANY   574 384 Steel
Locked Coil   Fan   1   -   -
-    -   Ref 28,66,75
-

77 **NORDERELBE**   1962 Hamburg   WEST GERMANY   564 210-210 Steel
Locked Coil   Star   1   -   -
-    -   Ref 28,61,65,66
Sockets loosened,twisted on stay

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

39 HOECHST BRIDGE 1972 FRANKFURT (Main) WEST GERMANY 486 129 Concrete
Parallel Bars threaded Harp 2 FARBWERKE HOECHST AG. Dyckerhoff & Widmann AG.
- YES Ref 18,19,28,36,37,129
-

139 NECKARCENTER FOOTBRIDGE 1975 Mannheim WEST GERMANY 460 - Concrete
Parallel wire PE grout - - - Leonhardt
- - Ref 140,BBR
54No7mm BBRV

62 LUDWIGSCHAFEN 1968 Hochstrasse, Ludwigschafen WEST GERMANY 453 453 Steel
Locked Coil Radiating 2 inc - Rheinstahl
- - Ref 28,66,71,75
-

32 FRANKLIN BRIDGE 1974 Dusseldorf WEST GERMANY 410 135-69 Steel
Locked Coil Single Stay 1 - -
- - Ref 28
-

50 JULICHER STR 1963 Dusseldorf WEST GERMANY 325 105-105 Steel
Locked Coil Single Stay 1 - -
- - Ref 28
-

44 SCHILLERSTRASSE FOOTBRIDGE 1961 Stuttgart WEST GERMANY 226 - -
Parallel wire PE grout - - - -
- - Ref 75,140,BBR
90No6mm BBRV

137 BICKENSTEG FOOTBRIDGE 1973 Villingen WEST GERMANY 220 - -
Parallel wire PE grout - - - -
- - Ref 140,BBR
85No7mm HiAm

168 GROWIAN 2 1981 Bremerhaven WEST GERMANY ? - -
Parallel wire PE grout - - - -
- - BBR
31No7mm HiAm

No. Name. Date. Location. Country. Span. Side-Spans. Deck-S/C.
Cables. Elevation. Planes. Owner. Designer.
Testing. Questionaire-Reply. References.
Comments.

78 NOVI SAD 1981 - YUGOSLAVIA 1152 - Steel
Parallel wire PE grout Fan - - Sloboda
- - Ref 140, INT. CONSTRUCTION Apr'86,BBR
295No7mm HiAm

95 SAVA RIVER 1978 Belgrade YUGOSLAVIA 835 - Steel
Parallel wire PE grout - - - Hajdin
- - Ref 37,140,179,BBR
290No7mm HiAm

75 LUANGWA 1968 - ZAMBIA 728 131-131 Concrete
- Fan + Star 2 - Freeman,Fox & Partners (RDL)
- - Ref 28
-

APPENDIX B

APPENDIX B

QUESTIONNAIRES
DATABASE

This appendix contains a list of bridges for which significant information was obtained.

Q-01, Q-02 and Q-D1 refer to the information sought and copies of the questionnaire form sheets are included in this Appendix.

Bridge	Country	Q-01	Q-02	Q-D1	Drawings	Specificatio
Airport Hotel	HONG KONG	YES	YES	YES	YES	-
Annacis	CANADA	YES	YES	-	YES	YES
Batman Bridge	AUSTRALIA	-	-	-	YES	YES
Baytown	USA	YES	YES	-	YES	YES
Benton City	USA	YES	-	-	YES	-
Burlington	USA	YES	YES	YES	YES	-
Chao Phya	THAILAND	YES	-	-	YES	YES
Chung Yang	TAIWAN	-	-	-	YES	-
Cochrane	USA	YES	-	-	-	-
Cooper River Bridge	USA	YES	-	-	-	-
Dame Point	USA	YES	-	YES	YES	YES
Deggenau Bridge	WEST GERMANY	YES	-	-	YES	-
Dolsan Bridge	KOREA	YES	YES	YES	-	-
Erskine	GREAT BRITAIN	YES	YES	YES	YES	-
Hawkshaw	CANADA	YES	YES	-	YES	-
Hoechst Bridge	WEST GERMANY	YES	-	-	-	-
Huntington East	USA	YES	YES	YES	YES	YES
James River	USA	YES	-	-	YES	YES
Jindo Bridge	KOREA	YES	YES	YES	-	-
Kwang Fu	TAIWAN	-	-	-	YES	-
Luling Bridge	USA	YES	YES	YES	YES	YES

Bridge	Country	Q-01	Q-02	Q-D1	Drawings	Specification
Moore (Capt.William)	USA	YES	YES	YES	YES	YES
Myton Bridge	GREAT BRITAIN	YES	YES	YES	YES	YES
Neches	USA	YES	YES	-	YES	-
Pasco - Kennewick	USA	YES	YES	YES	YES	YES
Penang	MALAYSIA	-	-	YES	YES	-
Quincy Bridge	USA	YES	-	-	YES	YES
Roosevelt Lake	USA	YES	YES	-	-	-
Sacramento River	USA	-	-	-	YES	-
Sitka	USA	YES	YES	YES	YES	YES
Sunshine Skyway	USA	YES	YES	-	YES	YES
Talmadge,Memorial	USA	YES	YES	YES	YES	-
Weirton-Steubenville	USA	YES	YES	-	YES	YES
Wye	GREAT BRITAIN	YES	YES	YES	YES	-
Clark	USA	YES	-	-	-	-
Cotton Tree Drive	HONG KONG	YES	-	-	YES	-
Menomonee Falls	USA	YES	YES	-	YES	-

QUESTIONNAIRE FORM SHEETS

PRIVATE & CONFIDENTIAL

QUESTIONNAIRE
for Owners of Cable-Stayed Bridges

Introduction

We have completed parts of this questionnaire where we think we already have the information. Can you please check that our entries are correct and alter or expand upon them as necessary.

All information obtained from the responses to this questionnaire and possible further discussion with the designers and contractors will be treated in strictest confidence.

Data will be presented in our report in such a form that identification of the particular bridge source will not be obvious to the reader. Topics will be discussed in general terms and grouped together under common headings (such as:- form of cable used, form of cable socket, form of attachment to the structure, etc.). They will not be related to countries or to individual bridges.

In this way total confidentiality will be ensured. Over and above this, where those supplying data express certain wishes regarding confidentiality these will be fully respected.

Please note, that it is of great importance for this project and for future effective design of bridges round the world that as much information as possible should be offered from the practical experience observed on existing bridges. There is always a limit to the usefulness of theoretical and/or laboratory work if there is not the practical background to provide effective calibration.

1. <u>BASIC DATA</u>

a) Name of Bridge: ....

b) Location: ....

c) Completion date: ....

d) Bridge Owner - Authority: ....
Tel.No. (...)................
Address: ....
....
Name of Maintenance Engineer: ....
(address and telephone number if different from above)
....

e) Design Engineer - Firm: ....
Tel.No. (...)................
Address: ....
Name of Engineer responsible for this bridge: ....
(address and telephone number if different from above)
....
Name of Resident Engineer during construction: ......
....
(address and telephone number if different from above)
....

f) Main Contractor - Firm: ....
Tel.No (...)..................
Address: ....
....
Name of the Agent ....
(address and telephone number if different from above)
....

g) Subcontractors (please give a list together with their fields of activity particularly those concerned with erection and fabrication of cables ....
....
....

h) As-built Drawings:

Will you please supply as-built drawings giving:-

(i) a general arrangement of the bridge

(ii) a dimensioned elevation and cross-section

(iii) dimensioned details of the cable-stays, their sockets and attachments to the superstructure and towers (saddles, anchorage beams, cable-stay impact protection devices, oscillation damping devices, cable clamps etc.)

(iv) Where modifications have been carried out since production of these drawings please give details

i) Specification:

Please give details of the specification provided for the design and construction of the cables and sockets.

Please may we have your early response to this section, together with any drawings that you can supply. Your co-operation on this will be greatly appreciated and will help the speedy conclusion of this project.

PRIVATE & CONFIDENTIAL

SECOND QUESTIONNAIRE
for Owners of Cable-Stayed Bridges

Introduction

We have completed parts of this questionnaire where we think we already have the information. Can you please check that our entries are correct and alter or expand upon them as necessary.

All information obtained from the responses to this questionnaire and possible further discussion with the designers and contractors will be treated in strictest confidence.

Data will be presented in our report in such a form that identification of the particular bridge source will not be obvious to the reader. Topics will be discussed in general terms and grouped together under common headings (such as:- form of cable used, form of cable socket, form of attachment to the structure, etc.). They will not be related to countries or to individual bridges.

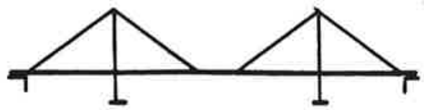
In this way total confidentiality will be ensured. Over and above this, where those supplying data express certain wishes regarding confidentiality these will be fully respected.

Please note, that it is of great importance for this project and for future effective design of bridges round the world that as much information as possible should be offered from the practical experience observed on existing bridges. There is always a limit to the usefulness of theoretical and/or laboratory work if there is not the practical background to provide effective calibration.

1. GENERAL DESIGN DATA

a) Bridge Spans: (metres or feet)...................................

b) Bridge type:-
(please circle
(as appropriate)



- single stay

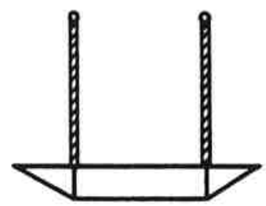
- harp arrangement

- fan arrangement

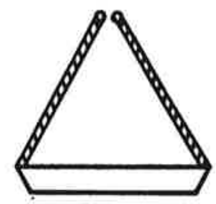
- Radiating arrangement

- Single main span (as shown in sketches above)
- More than one main span

- one plane of stays



- 2 planes (vertical)



- 2 planes (inclined)

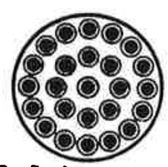
- others (details please)

c) Composition of Main Bridge Girder/Girders - Steel - Concrete
(please circle as appropriate) - Composite

d) Tower Construction - Steel - Concrete

e) General arrangement of each cable-stay (i) length (ii) diameter and number of cables making up each cable-stay ....

f) Cable type:-
(please circle as
appropriate)

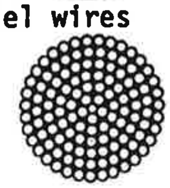


Parallel bars

Parallel wires

Parallel Strands

Helical/locked-coil strand



Spiral Bridge Strand

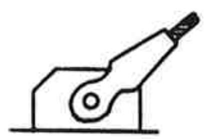
Others (details please)

g) Cables manufactured in:- USA, Europe, Japan, Other
(please circle as appropriate)

h) Cable end anchorages - Type of sockets (description or sketch) ....

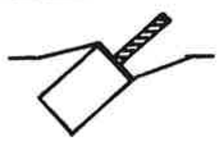
- Anchoring method
  - white metal
  - zinc
  - resin with steel balls
  - wedge grip
  - button head
  - other (details please)

i) Connection between cable anchorage and main structure



- simple pin

- universal joint

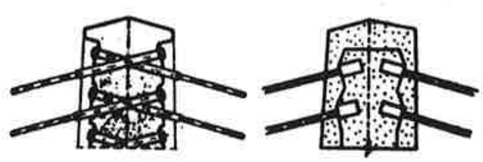


- direct bearing

- other (details please)

j) Connection provided at towers

- saddle
- pin (as in (h) above)
- other (details please)



- direct anchorages
- universal joint (as in (h) above)

k) Means of cable stay adjustment

- shim stack
- none
- recessible anchorage
- other

l) Amount of stay tension that can be relieved by adjusting the anchorage? ....

2. TESTING OF LENGTHS OF CABLE AND COMPLETE STAYS

a) Testing Agencies:-
   - For lengths of cable (wire, strand, bar, rope) ....
   - For complete stays ....

b) Type of Testing (ultimate, fatigue, both)
   - For cables ....   - For complete Stays ....

c) Was lateral bending applied during any test? ....

d) Fatigue Test Parameters (number of cycles ....., stress range ....., bending stress ......, definition of failure ......, length of specimens ......)

e) Any problems encountered? ....

3. LOADING

a) Design Traffic Capacity - No. of lanes of highway traffic ....
   - No. of rail or metro tracks ....
   - No. of cycle tracks or footways ....

b) Estimated/Recorded Traffic flow since bridge opening
   - 1st year ....
   - 2nd year ....
   - Most recent 12 month period .....

   (please add intervening years if traffic flow does not fall into a roughly pro-rata increase).

c) Description of Traffic flow - Please give typical make up of traffic flow, particularly the percentage of vehicles weighing more than 20 tons ....

d) Seismic Risk zone .... Has there been seismic activity at the bridge site since construction? ....

e) What maximum and minimum temperatures have been recorded at the site or on the bridge? ....

f) Have any steps been taken to minimise temperature differential between the cable-stays and the deck (colour, sheathing etc.)?
   ....

g) Do the cable-stays ice in certain conditions?
   If so please give details of amount, frequency and effect on dynamic behaviour ....

4. CORROSION PROTECTION OF THE CABLE-STAYS

a) What atmospheric pollution exists at the bridge site? (Industrial, salt laden from the sea, etc.) ....

b) Were the individual wires originally galvanised? ....

c) Was internal lubrication or corrosion protection applied during the cable manufacturing process? (If so please give details)

....

d) Was external corrosion protection applied after original manufacture? (If so please give details) ....

e) Was an external sheath provided? (If so please give details including details of the sealing agent or grout used between cable and sheath)

....

f) Has the original corrosion protection system been modified? (If so please give details) ....

g) Details of the planned maintenance programme -

- How often are the cables and sockets inspected? ....
- What method of inspection is used (Visual, various non-destructive testing methods) ....
- Do the sockets receive regular attention? .... (details please - paint, sealant, grease etc.) ....
- Is it possible to visually inspect the region of the cable where it enters the mouth of the sockets? ....
- Have any maintenance services been performed to enhance corrosion protection? ....

5. POSSIBILITY OF CABLE-STAY REPLACEMENT

a) Would the replacement of a cable-stay be:-
   (i) Straightforward? (with no traffic restrictions)
   (ii) Inconvenient? (with some traffic restrictions - please give details)
   (iii) Seriously inconvenient? (with complete bridge closure)
   (iv) Uneconomic? (cheaper to build a new bridge)

b) Have any cable-stays been replaced and if so:-
   (i) please give reasons with dates and location on the bridge
   (ii) has there been any detailed examination and report on the removed stays? If so please give details.

6. OBSERVATIONS ON THE PERFORMANCE OF THE CABLE-STAYS

a) Do the cable-stays or the whole bridge itself oscillate in any wind conditions? ......
If so please give details (wind speed and direction relative to bridge, frequency, amplitude, mode shape etc.)

b) Do the cable-stays or the bridge itself oscillate in any live load conditions? ...... If so please give details

c) Has any artificial damping system been added at the design stage or later? ...... If so please give details (hydraulic dampers, spring dampers, cable clamps etc.)

d) Has any deterioration of the cable-stays been reported? .... (Corrosion, wire breakages, socket draw etc.)
If so, please give dates and details.

e) Have any cable-stays been damaged from traffic impact? ....
If so please give details.

f) Do the cable stays have local protection devices against traffic impact? ...... If so do these make the cable locally more stiff in this zone, and do these make visual inspection in the region of the socket mouth difficult?

g) Is de-icing road salt used? ....

h) Does road surface water reach the cable-stays and/or the sockets? .... (either through splashing or directly)

i) Is there provision for lubrication of the pins? ....
If so does it effectively allow rotation at the pins?

j) Have any other problems, either of a structural or detailing nature, been reported or suspected? ......
If so please give details and indicate the remedies used or proposed.

k) Are there any details relating to the sockets, cable-stays and attachments to the structure that might be improved to reduce the risk of corrosion and fatigue? ....

i) Have any studies or tests been undertaken on the bridge in relation to performance of the cable stays?

7. GENERAL QUESTIONS

a) In the close location of the cable-stayed spans are there steep gradients, traffic lights, roundabouts or toll collection areas which might tend to locally intensify traffic loading?

b) Have any studies or tests been undertaken, either on the bridge itself or in laboratories, in relation to fatigue of cable-stays, their connectors, or their corrosion protection? ....
If so please give details.

c) Bearing in mind the objectives of this exercise, do you think that there are any other factors which would be of interest? ....
If so please give details.

PRIVATE & CONFIDENTIAL

QUESTIONNAIRE
for Designers of Cable-Stayed Bridges

Introduction

We have completed parts of this questionnaire where we think we already have the information. Can you please check that our entries are correct and alter or expand upon them as necessary.

All information obtained from the responses to this questionnaire will be treated in strictest confidence.

Data will be presented in our report in such a form that identification of the particular bridge source will not be obvious to the reader. Topics will be discussed in general terms and grouped together under common headings (such as:- form of cable used, form of cable socket, form of attachment to the structure, etc.). They will not be related to countries or to individual bridges.

In this way total confidentiality will be ensured. Over and above this, where those supplying data express certain wishes regarding confidentiality these will be fully respected.

Do please note, that it is of great importance for this project and for future effective design of bridges round the world that as much information as possible should be offered from the practical experience obtained on existing bridges. There is always a limit to the usefulness of theoretical and/or laboratory work if there is not the practical background to provide effective calibration.

1. BASIC DATA

a) Name of Bridge: ....

b) Location: ....

c) Completion date: ....

2. DESIGN CODES:-

a) What design codes were used? i) for loading? ....

ii) for structural analysis (including fatigue of cable stays and factors of safety for wires ropes)? ....

....

b) Were any omissions by the codes noted and if so how were they covered? (particularly fatigue and corrosion of the cable stays)

....

c) What were the cable-stay design parameters?

i) allowable stresses ....

ii) allowable stress ranges ....

3. LOADING:-

a) What traffic loading was the bridge designed to carry (code used - or full description)? ....

b) How many traffic lanes is the bridge designed to accommodate? ......

what is their width? ....

and transverse location? ....

c) What is the design wind speed? ....

d) What maximum wind speed has been recorded at the bridge? ....

e) What is the approximate dead weight of the main bridge girder/girders per metre length (per foot length) ....

f) What were the time dependent design parameters?

i) temperature ....

ii) shrinkage ....

iii) creep ....

4. CABLE STAYS:-

Please supply details of:-

a) cross sectional shape of cable (wire,strand,bar,rope)...............
b) metallic sectional area ....
c) lay length ....
d) ultimate tensile strength of wire used ....
e) the specification chosen for the wire (strand,bar,rope) used in the cable-stays ......
f) manufacturers minimum breaking load ....
g) the modulus of elasticity of each cable ....
h) the effective second moment of area of each cable ....
i) internal lubrication ....
j) wire galvanizing (please indicate if wires are not galvanized) .....
k) external corrosion protection ....
l) external sheathing, if employed ....
m) sealing between cable and sheath (grout, grease etc.) ....
n) cable prestretching before erection (load, time, number of cycles, etc.) if relevant (please indicate if cables were not prestretched)
o) the dead and superimposed dead load carried by each cable ....
p) the maximum design range of live load in each cable ....
q) the accuracy of tension determination in individual strands where more than one strand was anchored at an anchorage. ....

5. SOCKETS:-

Please supply details of:-

a) type of socket employed ....
b) method of anchoring cable in socket ....
c) method of attaching socket to structure ....
d) maximum theoretical inclination of cable relative to socket centre line in both longitudinal and transverse senses. ....

6. SADDLES:-

a) Where the cable-stays change direction (over a saddle or other method) what minimum radius does the cable follow? ....

b) Has any special edge detail been provided at the mouth of the cable trough (radiused corner etc.) ....

7. GENERAL

a) Have any other problems, either of a structural or detailing nature, been reported or suspected? If so please give details and indicate the remedies used or proposed for:-

- sockets
- cable stays
- corrosion protection
- wind excited oscillations
- temperature effects
- any other effects.

b) Have any studies or tests been undertaken, either on the bridge itself or in laboratories, in relation to fatigue of the cable-stays, their connectors, or their corrosion protection. If so may we have details please? ....

c) Are there any other studies or tests whose results you may have used and found particularly helpful? ....

APPENDIX C

APPENDIX C

BIBLIOGRAPHY
(Including selected abstracts covering fatigue strength)
DATABASE

This appendix contains a list of the research reports, papers and articles that have been collected and is presented by author in alphabetical order.

A list is included of the key words used, together with a description of the subjects covered by each word.

Selected abstracts of twenty papers covering fatigue strength are also included in this appendix in order to provide an over-view of the quality of material that forms the foundation of this project.

KEY WORDS USED IN THE BIBLIOGRAPHY

(NOTE: "Fatigue" does not occur below because almost all documents cover some aspect of fatigue)

DESIGN	- Codes - Direct guidance - General bridge configurations - Manufacturer's brochures
TEST	- Test results - S-N Curves - Research Reports - Theoretical approach to Fatigue
BRIDGE	- Details of bridges given - cross referenced to bridge data base
WIND	- Aerodynamic effects
LOADING	- Traffic surveys - Theoretical Models of traffic flow
CABLE	- Information on Cable-Strand - Parallel Wire
ROPE	- Information relating to Wire Ropes but still relevant
SOCKET	- Information on Sockets
PROBLEM	- Corrosion, Vibration, Detection Methods
LOCALISED	- Stresses in Saddles, Clamps, Socket mouth etc.
SPECIFICATION	- Material Details, Surface finish of wire

EXTRAPOLATE	**- Tests on short lengths of cable or single wires related to full size cables**
DATA BASE	**- Searches in Data Bases**
BOOK	**- Books on cable-stayed bridges covering most of the above subjects**

SELECTED ABSTRACTS COVERING FATIGUE STRENGTH

1. MATSUKAWA, KAMEI, FUKUI, SASAKI. "Fatigue resistance analysis of parallel wire strand cables based on statistical theory of extreme"

The effects of wire length, wire diameter, number of wires and minimum stress on the fatigue strength of parallel wire cables are examined statistically with reference to existing test data. The results are used to present a formulation for the fatigue strength of HIAM cables. It is concluded that more data is needed to confirm the fatigue limit of such cables.

2. MAKINO. "Projekt einer Schragseilbrucke uber den Yodo-Fluss in Osaka".

The Yodo River bridge project is described. The steel superstructure is supported by a modified fan arrangement of parallel wire cables consisting of almost 400 7mm wires each. Fatigue test results for plain and galvanised wires are given.

10. FHWA/RD-81/090. "Analytical study for fatigue of highway bridge cables"

The wind vibration behaviour of stay cables is discussed. The geometrical and structural characteristics of parallel and spiral wire cable are discussed. Examples of calculated wind velocities, cable frequencies, amplitudes and bending stresses are given for actual bridges. The fatigue behaviour of wires and cables is formualted from first principles. Fracture mechanics is used to determine crack growth and final fracture criteria. The effect of initial defect size on life is calculated for single wires. Comparison of single wire and rope behaviour is discussed briefly.

15. HOBBS, SMITH. "Fatigue performance of socketed terminations to structural strands."

The results of a programme of fatigue testing of 16mm and 38mm diameter spiral strands are summarised. Tests were done axially and also with a transverse load in air. Wire failures occurred in the outer wires first near the sockets. Initial wire failures occurred early on in life. Mean stress did not appear to be an important parameter in determination of life. Damaging was measured under cyclic loading.

16. HOBBS, SMITH, BOLTON, WALLACE. "Discussion" on Ref.15 above.

Design implications are discussed. Improvement of design and use of dampers to reduce transverse vibrations recommended. Socket design discussed.

21. HOBBS, GHAVAMI. "Fatigue of socketed cables"

Results of 3 axial and 6 transverse fatigue tests on 38mm diameter spiral strands (see Ref. 15). Fretting of crossing wires between layers under bending action is identified as an important consideration.

29. HOBBS. "Fatigue of socketed cables, tests on 16mm specimens"

Results of axial fatigue tests on 12 no. 16mm diameter white metal socketed spiral strands are described in more detail than in Ref. 15. Details of fractured wires shown.

85. HOBBS, GHAVAMI "Fatigue of structural wire strands"

Summary Paper reviews work in Refs. 21 and 29.

32. TILLY "Design and performance of stay cables"

This is one of the most up-to-date review papers on cable stay bridge practice (1986). Current practice on cable type, terminations, and cable arrangements. Methods of corrosion protection and in service inspection are discussed including methods of measurement of structural behaviour. Case histories of wire failures on 8 structures are described. 7 British cable stayed bridges are described. Structural damping and methods for damping wind oscillations of cables themselves are discussed.

59. FLINT & NEILL PARTNERSHIP. "Investigation of fatigue failure in socketed terminations on structural cables".
Analysis of fatigue test data on 16mm and 38mm diameter spiral strand (Refs. 21 and 29). In particular a method for estimating the effects of bending moment at the socket under transverse vibrations is given.

79. CHAPLIN, SHARMAN. "Load transfer mechanics in resin socketed terminations".

The mechanics of resin sockets for wire ropes are discussed. Tests are carried out to support the theory of load transfer through the resin to the socket walls. The roles of friction, slip of the resin block and bursting pressure on the socket are discussed.

89. GATHMAN "Resin socketing for wire rope attachments"

The advantages of resin socketed rope terminations over zinc and white metal socketing media are discussed. The importance of following strict procedures for socketing is emphasised.

116. RAOOF, HOBBS "The contrasting properties of two spiral strands".

Theoretical calculation of the axial and bending response for 53 and 63mm diameter spiral strands is described using analysis methods described in other references by the authors. Cyclic energy dissipation is calculated. Maximum wire stresses and interlayer fretting movements are also calculated.

117. HOBBS, RAOOF "The behaviour of large spiral strands"

The theoretical behaviour of large spiral strands is reviewed. Parameters affecting elastic axial modulus histeresis and torsional stiffness are discussed with reference to experimental data. Bending effects are briefly mentioned. The importance of slip or no-slip between wires in determining behaviour is emphasised.

118. RAOOF, HOBBS "The bending of spiral strand and armoured cables close to terminations"

Theoretical models for detailed behaviour of wire interaction in multi-layer spiral strand is given. This gives an explanation for the wire breakages in the core of the strand under bending action at the sockets. An analysis based on the model is used to explain the results of tests on 38mm spiral strand (Ref.21).

120. KOBE (SHINKO WIRE) "Report on the fatigue test of 0.250 inch single wires with surface flaws".

Investigation into the effects of surfaces flaws on the fatigue strength of 0.250 inch diameter prestressing wire described. 24 tests were done on wires with various types of visible surface flaws. Fatigue tests at stresses between 33% and 47% of guaranteed ultimate tensile stress of the wire all survived $2x10^6$ cycles without failure. Micro sections of the various flaw types are shown.

121. DYKERS, FRANK "Report on test of Luling hybrid socketed cable stays"

Axial test results are given for 4 HIAM socketed tendons. Wire numbers varied from 103 to 307. 2 tendons were fatigue tested at room temperature in the range from 35% to 45% of the ultimate tensile strength, one of which was subsequently tested to total failure statically. The other 2 tests were creep tests with socket temperatures up to 188°F followed by static tests to failure. The locations of wire fractures and cycles to failure were recorded. Displacements of button heads and end plates were also recorded. Detailed mechanical test results on wire and socket materials are given.

126. TAMHANKAR "Design of cables and their anchorages in cable-stayed girder bridges"

This is a review paper and does not give any new test data. The merits of locked coil and parallel wire strands are discussed. Design of sockets and saddles are discussed with reference to German practice up to 1976.

177. BBR "Cable-stayed structures"

Axial fatigue tests results are given for 72 ungrouted HIAM and DINA parallel wire socketed tendons, sizes of tendon varied from 7 wire to 307 wire with a wire diameter of 7mm. The upper stress levels varied from 17% of the guaranteed ultimate tensile strength to 72%. All tests were done at constant amplitude. Stress ranges varied from 147 N/mmâ to 540 N/mmâ. The number of cycles to first wire failure and also to 5% area loss were noted. A number of tests had run-outs at 2 million cycles without reaching either the 5% area loss or in some cases even single wire failure. In 35 cases the fatigue tests were followed by a static test to failure. No tests in corrosive conditions are reported. Details of test set up not given.

200. BASU, CHI. "Analytical study for fatigue of bridge cables."

The effects of wind excitation on bending stress is discussed. Fracture mechanics analysis of cracks in wires leads to S-N curves for wires with cracks of different initial sizes. The paper claims to determine the fatigue behaviour of complete cables from the study of the behaviour of single wires.

201. BIRKENMAIER, NARAYAN "Fatigue resistance of large high tensile steel stay tendons"

The statistical analysis of fatigue tests on 210 wire specimens from the Sava Bridge Project is discussed. Details of the test up are not given. Reference is made to tests which show the serious effect of corrosion. Tests in sea water showing a strength of only 25% of that in air. The results of 5 fatigue tests on HIAM and DINA anchored tendons (19 to 295 wire) are given. Recommended design S-N curves are given for wires and tendons, the latter being 140 N/mmâ below that for wires. It is suggested that the design fatigue strength should be baesd on the endurance at which a 5% loss of static strength has been reached.

Author
Title
Key-Words Source

AASHTO. 209
Standard Specifications for Highway Bridges.
Design 1983 (+Mods 1984 & 1985).

AIMONE. 174
Non-Destructive Wire Rope Tester.
Problem

AIMONE. 175
Non-Destructive Testing of Wire Ropes in the Mining Industry
Problem Paper N.66.

ANDRA und SAUL. 196
Tests on parallel wire and strand cables for North Bridge..
GERMAN. Test Die Bautechnik 9,10&11/1974

ANDRA und ZELLNER. 197
Parallel Wire Bundles and their Anchorages.
GERMAN,Socket,Test Die Bautechnic 8&9/1969

ANNALS OF N.Y. ACAD.OF SCI. 160
Long-Span Bridges.
Book V.352.

ANON., 157
Brochures in Japanese.
JAPANESE. Bridge

ANON. 183
Acoustic Emission.
Problem

APEL, NUENNINGHOFF. 83
Improving Props.of H.T. Wire Ropes by Opt.of Constr. - Pt.1.
Rope,Test Wire, V.33,N.2, Mar/Apr.1983.

BAHKE. 86
Principles Defining the Strength of Wire Ropes & Chains-Pt.1
Test Wire, V.30,N.2, Mar/Apr.1980.

BARTLE, MUDGE. 122
Acoustic Pulsing - a Technique for Remote Defect Monitoring.
Problem The Welding Int.Res.Bull;V.24,Sep.1983

BASU, CHI. 11
Analytical study for fatigue of highway bridge cables.
Test FHWA/RD-81/090 Report July 1981

BASU, CHI. 200
Analytical Study for Fatigue of Bridge Cables.
Extrapolation, Wind IABSE Reports Vol.37.

BBR. 177
Cable-Stayed Structures.
Design 3/11/86

BECKER. 90
On the Fat.Strength of Rope Wires .... Hoisting Ropes.
Rope,Spec.,Extrapol. Wire, N.2, Mar/Apr.1979.

BECKER. 98
Question of the Fatigue Strength of Steel Wires.
GERMAN.Specification Stahl und Eisen,V.92,N.18, 31 Aug.1972

BELL, KRUKAR. 224
Automatic Vehicle Monitoring Demonstration Project.
0 TRB.Annual Meeting Wash,D.C., Jan.1987

BIANCHI, SANSON. 186
Le Pont De Saint-Nazaire - Saint-Brevin.
FRENCH. Bridge Travaux No.490 - Janvier 1976.

BIRKENMAIER, NARAYANAN. 201
Fatigue Resistance of Large High Tensile Steel Stay Tendons.
Test IABSE Reports Vol.37.

Author
Title
Key-Words Source
BIRKENMAIER. 163
Fatigue Resistant Tendons for Cable-Stayed Construction.
Cable,Socket IABSE Proc.P-30/80, May 1980.
BISHOP. 212
Enclosure - an alternative to Bridge Painting.
0 Research Report 83,ISSN 0266-5247,1986
BOWERS. 123
Loading History Yellow Mill Pond Bdge,Bridgeport,Connecticut
Loading Research Project HPR 175-332, May 1972
BRAULT, MATHIVAT. 187
Le Pont De Brotonne.
FRENCH. Bridge Travaux No.492 - Fevrier 1976.
BRITISH CODE BS 5400 Pt.10:1980. 210
Code of Practice for Fatigue.
Design British Standards Institution.
BRITISH ROPES. 44
Report on an Examination of Fractured Suspender Wires.
Test Letter to County of Avon, 3 Feb.1975.
BRITISH ROPES. 45
Report on Attempts to Measure Hanger Tensions.
Test Letter to Avon CC, 9 Jun.1975.
BSI. 181
BS 5400:Part 10:1980. Code of practice for fatigue.
Design
BURDEKIN, GEAREY. 173
Non-Destructive Methods for Field Inspection of Embedded ...
Problem UMIST/CAPCIS 851214, May 1986.
CASEY, WEDLAKE, TAYLOR, HOLFORD. 77
Acoustic Detection of Wire Rope Failure.
Problem Wire Industry, V.2,N.617, May 1985.
CASTILLO, CANTELI, ESSLINGER, THURLIMANN. 164
Statistical Model for Fat.Analysis of Wires,Strands & Cables
Test IABSE Proc.P-82/85, Feb.1985.
CBA - BUCKLAND, TAYLOR. 134
Annacis Cable Stayed Bridge.
Bridge Construction Progress Rpt.N.2,Sep.1985
CHAPLIN, SHARMAN. 79
Load Transfer Mechanics in Resin Socketed Terminations.
Socket Wire Industry, V.51,N.610, Oct.1984.
CHODAI Co.Ltd. 41
Brochure on Company - Chodai Co.Ltd.
Bridge
COHEN, BIRDSALL. 36
Long-Span Bridges Annuals of the N.Y. Acad.of Science,V.352.
Book 1980
COMPUTER. 23
RETREIVAL from 'COMPENDEX' Early Search May 1985.
Data BaseCABLES FAT. WELDING INST. 29 May 1985.
COMPUTER. 24
RETREIVAL from 'COMPENDEX' via DIALOG.
DaBa WIRE ROPES FAT. WELDING INST. 5 Feb.1986.
COMPUTER. 25
RETREIVAL from 'COMPENDEX' via DIALOG.
DaBaWIRE ROPES CORRO WELDING INST. 5 Feb.1986.
COMPUTER. 26
RETREIVAL from HRIS FILE SEARCH.
Data Base FATIGUE. NCHRP HRIS: Mar.1982, Oct.1983.

Author
Title
Key-Words Source
COMPUTER. 27
Almost Complete Duplicate of Ref.26.
Data Base FATIGUE. from NCHRP.
COMPUTER. 182
RETREIVAL from "Compendex".
Data Base The Welding Institute, 17 Oct.1986.
CONFERENCE ON BRITISH DESIGN RULES. 206
Bridge Aerodynamics.
Wind, Book 25-26 March 1981, T.Telford Ltd.
CONSTRUCTION TODAY. 133
Annacis Learns from Tacoma.
Bridge Nov.1985.
CONSTRUCTION TODAY. 137
Parana Heads for Home.
Bridge Jun.1986.
CULLIMORE. 31
The Fatigue Strength of High Tensile Steel Wire Cable.......
Test PRE. 1973.
DEMERS, SIMONSEN. 67
Montreal Boasts Cable-Stayed Bridge.
Bridge ASCE, V.41,N.ST8, Aug.1971.
DRAGONE, ROSSETTI. 97
Recent Investigations into Wire Rope Fatigue.
Rope,Test Wire Industry, V.43,N.507, Mar.1976.
DYCKERHOFF, WIDMANN. 129
Hoechst Bridge/Der Bau der2.Mainbrucke der Farbwerke Hoechst
GERMAN. Bridge Sent with Questionnaire response
DYKERS, FRANK. 121
Report on Test of Luling Hybrid Socketed Cable Stays.
Test,Cable,Socket The University of Texas, Sep.1980.
EDWARDS, PICARD. 128
Fatigue Characteristics of Prestressing Strand.
Test,Localised ICE Proc; Sep.1972.
ENGINEERING NEWS RECORD. 135
Skyway's Deck Girder Sags.
Bridge,Problem 27 Mar.1986.
ENGINEERING NEWS RECORD. 208
Trolleys Cut Cost of Inspecting Bridge Cables.
Problem ENR, May 15,1986.
ENGINEERING NEWS RECORD. 225
Texas plans record bridge.
0 ENR January 30,1986.
ERNST. 114
E-Modiles for Cable Regarding Sagging.
GERMAN.Design,Cable Bauingenieur 40(1965), S.52-55.
FEDERAL HIGHWAY ADMINISTRATION. 10
Analytical Study for Fatigue of Highway Bridge Cables.
Wind,Test,Bdge,Extrp Jul.1981
FEDERAL HIGHWAY ADMINISTRATION. 14
Loading Spectrum Experienced by Bridge Structures in USA.
Loading FHWA/RD-85/012 Report Feb. 1985
FIELD. 56
Rpt.on Exam'tion of Fractured Wires from Sev.Bdge.Sus.Cables
Test 6 Feb.1975.
FIGG, MULLER. 124
Recent Developments in Concrete Cable Stayed Bridges.
Design

Author
Title
Key-Words Source
FIRMAGE. 66
Europe's Dramatic Cable-Stayed Bridges.
Design,Bridge ASCE, V.44,N.ST9, Sep.1974.
FISHER, ALBRECHT, YEN, KLINGERMAN, McNAMEE. 220
Fat.Strength o'Steel Beams w'Welded Stiffeners & Attachments
0 NCHRP 147, 1974.
FISHER, FRANK, HIRT, McNAMEE. 219
Effect of Weldments on the Fatigue Strength of Steel Beams.
0 NCHRP 102, 1970.
FISHER, YEN, WAGER. 223
Rev.o'Field Mea.for Distortion - Induced Fat.Cra.in Sl.Bdges
0 NCHRP Project 12-28 (6).
FLINT & NEIL Sponsor I.B.A. 20
Investigation of Fatigue Failure in Socketed Terminations.
0 Apr.1980
FLINT & NEILL Partnership. 59
Investigation of Fat.Failure in S'kd Terms.on Struct.Cables.
Test Apr.1980.
FRENKIEL. 95
REFMA - Ref.Machine - A New Era in Fat.Tests of Wire Ropes.
Rope Wire Industry, V.44,N.523, Jul.1977.
FREYSSINET INTERNATIONAL. 188
Cable Stays.
Test,Specification 12/11/86.
F.& NP. 47
Report on Test of Hanger CB 27-25U.
Test Letter to DTp, 26 Feb.1980.
GABRIEL. 78
On the Fatigue Strength of Wires in Spiral Ropes.
Test,Extrapolation ASME, V.107,N.1, Mar.1985.
GABRIEL. 203
Tension Members in Steel Wire Strand and Rope.
GERMAN, Test IABSE Reports Vol.37.
GAMBRELL. 100
Predicting Fat.Life of Wire Rope from Tests on Single Wire.
Extrapolate,Rope Wire & Wire Prods;V.145,N.11, Nov.1970
GARRIDO. 119
Design Fabr'tion & Ere'tion of C-S for the Luling.....Bridge
Cable,Bridge Trans.Res.Board...Wash'D.C,16 Jan.1985
GATHMAN. 89
Resin Socketing for Wire Rope Attachments.
Socket Wire Journal, V.12,N.6, Jun.1979.
GERMAN CODE DIN 1072. 168
Road and Foot Bridge Design Loads.
ENGLISH. Design Aug.1983. (Translation from German).
GERMAN CODE DIN 1073. 169
Steel Road Bridges, Design Base.
GERMAN. Design Juli 1974, Preisgr.10.
GERMAN CODE DIN 1073. 199
Steel Road Bridges, Bases of Design.
ENGLISH, Design Translation
GERMAN CODE DIN 18 809. 170
Steel Road and Foot-Bridges;Dimensioning,Design,Construction
GERMAN. Design Okt.1084, Preisgr.10.
GHOSN, MOSES. 215
Markov Renewal Model for Maximum Bridge Loading.
0 JEM,V.111,N.9,Sep.1985,ASCE PP.No19971

Author
Title
Key-Words   Source
GHOSN, MOSES.   216
Reliability Calibration of Bridge Design Code.
                 0   JSE,V.112,N.4,Apr.1986,ASCE PP.No20503
GIMSING.   33
Cable Supported Bridges Concept & Design 1983.
Book
GLASS.   125
Manufacture, Uses and Care of Steel Wire Ropes.
Rope   Founding,Welding,Prod.Eng.Jour;Nov1970
GOODYEAR.   139
Issues in Stay Cable Fatigue Design Loading.
Design   CIB Major Bdge.Conf.New York, Apr.1986
GRANT.   102
Pasco-Kennewick Bridge - The Longest C-S Bridge in N.America
Bridge   ASCE, Aug.1977.
GRANT.   161
The Pasco-Kennewick Intercity Bridge.
Bridge   PCI Journal, May/Jun.1979.
GRATTESAT.   184
Ponts de France.
FRENCH. Book,Bridge   Sep.1982,Presses Ponts et chausses
GURNEY.   189
Notes on Mechanics Colloquium.
Test   Cambridge, Nov.13,1986.
GURNEY.   190
Fat.Tests ... to Assess the Validity of Miner's ... Rule.
Test   Proc. Royal Society London A386 1983
GURNEY.   191
Fat.Tests on Fillet Welded Joints Under Simple Var.Amp.Load
Test   T.W.I.Research Report 182/1982 May82.
GURNEY.   192
Fat.Tests Under Variable Amplitude Loading.
Test   T.W.I.Research Report 220/1983 July83.
GURNEY.   193
Fat.Tests on Fillet Welded Joints Under Variable Amp.Loading
Test   T.W.I.Research Report 293/1985 Nov85.
GUR'EV, GOKHBERG, AVIDON.   108
The Corrosion Under Stress of Composite Wire.
Specification   Soviet Materials Sci;V17,N2,Mar/Apr'81
GUTE, PODOLNY, SEIM, TANG, ULSTRUP, YIU.   28
Bibliography and Data on Cable-Staying Bridges.
Bridge   ASCE,Proc; V.103,N.ST10,Oct.1977
GUTE.   68
Design and Construction of the Sitka Harbor Bridge.
Bridge   ASCE, NatStrEngMeeting, 9-13 Apr.1973.
GUTE.   150
First Vehicular Cable-Stayed Bridge in the U.S.
Bridge   Civil Engineering - ASCE, Nov.1973.
HANZAWA, YOKOTA, TODA, YOKOYAMA.   84
Fatigue Behavior of Large-Diameter Wire Ropes.
Rope,Test,Socket   SPEJ, V.22,N.3, Jun.1982, SPE 10905.
HARRIS.   185
The St.Nazaire Bridge.
FRENCH. Bridge   18 Mar.1976.
HEGAB.   211
Static analysis of cable-stayed bridges.
Design   ICE Proc.Pt.2, Paper 8961, Dec 1986.

Author
Title
Key-Words Source

HIGHWAYS & TRANSPORTATION. 176
Cable-Stayed Bridge in Spain.
Bridge H'WAYS & TRANS'TION,Jan84

HOBBS, GHAVAMI. 21
Fatigue of Socketed Cables.
Test Sep.1978

HOBBS, GHAVAMI. 85
Fatigue of Structural Wire Strands.
Test Int'nat'Jour.of Fat; V.4,N.2, Apr.1982

HOBBS, RAOOF. 117
The Behavior of Large Spiral Strands.
Cable Proc.5th Int'nat'Offshore.....NY 1986.

HOBBS, SMITH, BOLTON, WALLACE. 16
Discussion on Paper 8605 (Fatigue of Socketed Terminations).
Test ICE Proc.Pt.2, Paper 8605, Sep.1983.

HOBBS, SMITH. 15
Fatigue Performance of Socketed Terms.to Structural Strands.
Test ICE Proc.Pt.2, Paper 8605, Mar.1983.

HOBBS. 29
Fatigue of Socketed Cables,Tests on 16mm Specimens Imp.Coll.
Test Jun.1977

HOBBS. 30
Fatigue of Socketed Cables, A-8040 Insulator Tests Imp.Coll.
Test Feb.1979

HONSHU-SHIKOKU BRIDGE AUTHORITY. 132
Honshu-Shikoku Bridge Project, Kojima-Sakaide Route.
Bridge Jan.1986.

HUNTER (PETER FRAENKEL INTERNATIONAL LTD.) 127
Chao Phya Bridge, Bangkok. CABLES - Manufacture & Testing.
Bridge Works Visit,Jun.1986,CONTACT P.F.I.Ltd

IMPERIAL COLLEGE. 57
Fatigue of Socketed Cables. Tests on 16mm specimens.
Test Jun.1977.

IMPERIAL COLLEGE. 58
Fat.of S'kd Cables. In Line & T'vse Tests on 38mm Specimens.
Test Sep.1978.

JONES. 80
Behaviour of Wire Ropes in Offshore Applications.
Ropes Extrapolation Wire Industry, V.51,N.606, Jun.1984.

KAWECKI, OLEKSY. 82
Ultrasonic Testing of Steel Wire in Optimising Quality.
Test Wire Industry, V.51,N.604, Apr.1984.

KEATING, FISHER. 221
Fat.Behavior of Varible Loaded Bdge.Details Nr.the Fat.Limit
0 NCHRP Project 12-15 (5).

KEIL. 99
Time Analysis of Bi-Axial Stress Conditions .... Loads.
GERMANRope,Localised Drahtwelt, V.57,N.8, Aug.1971.

KERENSKY, HENDERSON, BROWN. 42
The Erskine Bridge Structural Engineers, Apr.1972, V.50,N.4.
Bridge

KIRK. 112
Marine Wire Rope Materials.
Rope,Specification Int.Ropeway Rev;V.12,N.2, Apr/Jun 1970

KNOX, HOMBERG, DEASON, McGIBBON, ET AL. 149
Discuss.Design and Construction of Kessock Bridge.
Bridge ICE Proc.Pt.1, Papers 8742-5, Feb.1986

Author
Title
Key-Words Source

KNOX, HOMBERG, DEASON. 147
Kessock Bridge: Design by Contractor.
Bridge ICE Proc.Pt.1, Feb.1984.

KOBE. 120
Rpt.on the Fat.Test of 0.250 Inch Single Wires with...Flaws.
Specification Shinko Wire Company Ltd. Nov.1980.

KOHLER, NURNBERGER. 202
Improvement o'Fat.Strength of Anchorages for Tendons & Ropes
GERMAN, Socket IABSE Reports Vol.37.

KRETSIS. 159
Lyne Railway Underbridge Over the M25.
Bridge ICE Proc.Pt.1, Nov.1982, V.72, P.8535.

LENGLET. 62
Brotonne Bridge - Longest Prestressed Conc.Cable Stayed Bdge
Bridge Eng.Series No.4, F.H.A., Jun.1978.

LEONHARDT, ANDRA und PARTNER. 140
Invest'tion of Cracks in the Poly'Pipes,LulingBdge.Louisiana
Bridge,Problem FINAL REPORT Jun.1986.

LEONHARDT, ZELLNER. 37
Cable-Stayed Bridges - IABSE SURVEYS S-13/80, May 1980.
Design,Bridge,Wind

LEONHARDT und ANDRA. 195
The concrete cable-stayed bridge...Pasco-Kennewick...USA.
GERMAN. Bridge (Leonhardt), 13/11/86.

LEONHARDT. 103
The Columbia River Bridge at Pasco-Kennewick, Washington,USA
Bridge Proc.8th Congress F.I.P;Pt.2, May 1978

LEONHARDT. 194
Brucken Bridge.
Book

LEONHARDT. 226
Cable Stayed Bridges.
0 FIP Congress 1986 New Delhi 16 Feb.

LEUTZBACH, WIEDEMANN. 138
Development and Applications of Traffic Simulation Models...
Loading TRAFFIC ENGINEERING + CONTROL,May 1986

LUCHT, DONECKER. 96
Factors Affecting Wire Rope Life in a Marine Environment.
Rope Offshore Conf.9th Annu.Tex;2-5 May1977

MAKINO. 2
Projekt einer Schragseilbrucke uber den Yodo-Fluss in Osaka.
Bridge,Test,Wind STAHLBAU 6/1985.

MATANZO. 109
Axial Fatigue of Wire Rope in Sea Water.
Rope,Test Offshore Conf.4th Annu.Tex;1-3 May1972

MATHIVAT. 104
The Brotonne Bridge.
Bridge Proc.8th Congress F.I.P;Pt.2, May 1978

MATSUKAWA, KAMEI, FUKUI, SASAKI. 1
Fatigue Resistance Analysis of Parallel Wire Strand Cables.
Test,Extrapolation Stahlbau 11/1985.

MATSUMOTO, TAKAHASHI, ODA. 111
The Omotogawa Bridge, a Prestr.Conc.C-S Bridge for Railway.
Bridge Jour.Jap.Prestr.Conc.Eng.AssocV24,1982

McGIBBON, BOOTH. 148
Kessock Bridge: Construction.
Bridge ICE Proc.Pt.1, Feb.1984.

Author
Title
Key-Words Source

MODJESKI & MASTERS. 12
The Mississippi River Bridge - Luling Louisiana.
Bridge

MOSES, GHOSN. 218
Instrum'tion forWeighingTrucks-in-Motion for H'way BdgeLoads
0 Aug.1983, Final Report FHWA/OH-83/001.

MOSES, SCHILLING. 222
Commentary on New Fatigue Design and Evaluation Procedures.
0 NCHRP Project 12-28 (3).

MURAKAMI, OKUBO. 74
Wind Resistant Design of a Cable-Stayed Girder Bridge.
Wind,Bridge IABSE 8th Congress, 9-14 Sep.1968.

NAKAYAMA. 38
Cable-Stayed Bridge with New Vierendeel - Type Girder.
Design,Wind Feb.1984 IABSE Proc. P-71/84.

NARITA, TADA, YOKOYAMA, SATO. 141
Characteristics of Aerodynamic Forces of Bridge Decks in....
Wind Univ.of Pittsburgh,Pennsylvania,Oct'85

NCB. 49
Test Report on New Rope Used in Autumn 1980 Contract.
Test Report 27 Aug.1980.

NCB. 52
52mm Dia.Strand Wire Samples. Test Report No.TD/MT/(82)26...
Test 26 Jan.1982.

NCE. 151
Morandi's Sleek 'Conc.Cable' Bdge.Opens the Rd.to Venezuela.
Bridge NCE, 18 Jul.1974.

NCE. 152
Truss Stresses Studied After Skyway Failure.
Bridge NCE, 23-30 Aug.1984.

NCE. 153
Structural Toll Mounts in Sub Zero Europe.
Problem NCE, 24 Jan.1985.

NCE. 154
Bangkok Builds to Beat the Jams.
Bridge NCE, 21 Feb.1985.

NCE. 155
Weld Faults Toppled Germans Mast Says Inquiry.
Problem NCE, 14 Mar.1985.

NCE. 156
Up and Under Cable Stay Keeps Low Profile.
Bridge NCE, 15 Aug.1985.

NEL. 54
Examination of Failed Suspender Cable Wires from the SevBdge
Test Report No.Z3/AUT/1.

NEW CIVIL ENGINEER. 136
Annacis Advances.
Bridge NCE 17 Oct.1985.

NYMAN, MOSES. 214
Calibration of Bridge Fatigue Design Model.
0 JSE,V.111,N.6,Jun.1985,ASCE PP.No19807

OPLATKA. 107
Wire Rope Termination Attachments.
Socket,Rope Wire Industry, V.50,N.600, Dec.1983.

OTAOLA. 3
Replacing Corroded Cables on a Cable-Stayed Bridge.
Problem,Bridge CIVIL ENGINEERING, Sep.1982.

Author
Title
Key-Words Source

PAULSON, FRANK, BREEN. 213
A Fatigue Study of Prestressing Strand.
0 Res.Rep.300-1,Proj.3-5-80-300,Apr.1983

PHOENIX, JOHNSON, McGUIRE. 217
Condition of Steel Cable after Period of Service.
0 JSE,V.112,N.6,June1986,ASCE PP.No20658

PODOLNY, FLEMING. 4
Historical Developments of Cable-Stayed Bridges(REPRINTof61)
Design,Bridge HIGHWAY FOCUS, V5,N2, Aug.1973

PODOLNY, FLEMING. 61
Historical Development of Cable-Stayed Bridges.
Design,Bridge Proc. ASCE, V.98,N.ST9, Sep.1972.

PODOLNY, KAVANAGH. 63
Discuss.of Historical Development of C-S Bridges by Podolny.
Design,Bridge ASCE, V.99,N.ST7,Proc;P.9826, Jul.1973

PODOLNY, SCALZI. 35
Construction & Design of Cable-Stayed Bridges 1976.
Book

PODOLNY, WOODS. 5
Discussion, Historical Developments of Cable-Stayed Bridges.
Design,Bridge HIGHWAY FOCUS, V5,N2, Aug.1973

PODOLNY, WOODS. 64
Discuss.of Historical Development of C-S Bridges by Podolny.
Design,Bridge ASCE, V.99,N.ST4,Proc;P.9640, Apr.1973

PODOLNY. 6
Cable-Stayed Bridges of Prestressed Concrete.
Design,Bridge HIGHWAY FOCUS, V5,N2, Aug.1973

PODOLNY. 8
Cable-Stayed Bridges and Wind Effects.
Wind,Bridge HIGHWAY FOCUS, V5,N2, Aug.1973

PODOLNY. 9
Economic Comparisons of Stayed Girder Bridges.
Design HIGHWAY FOCUS, V5,N2, Aug.1973

PODOLNY. 19
The Evolution of Concrete Cable-Stayed Bridges.
Design,Bridge CONCRETE INTERNATIONAL Aug.1981.

PODOLNY. 70
Cable-Stayed Bridges - A State of the Art.
Design,Bridge ASCE,NaWaResoEngMeeting,11-15 Jan.1971

PODOLNY. 71
Design Considerations in Cable-Stayed Bridges.
Design,Bridge ASCE, Conference St.Louis, 1974.

PODOLNY. 75
Cable Connections in Stayed Girder Bridges.
Design,Bridge AISCE JOURNAL 4th Qtr.1974.

PODOLNY. 93
Commmentary on the Tentative Recom'tions for C-S Bdge.Struct
Design ASCE,Jour.of the Str.Div;V103,N5,May77

PODOLNY. 94
Tentative Recommendations for Cable-Stayed Bridge Structures
Design ASCE,Jour.of the Str.Div;V103,N5,May77

PODOLNY. 180
U.S. Cable-Stayed Bridges - List.
Bridge 14/1/86

POLDOLNY, FLEMING. 7
Cable-Stayed Bridges -- Single Plane Static Analysis.
Design HIGHWAY FOCUS, V5,N2, Aug.1973

Author
Title
Key-Words Source

POST TENSIONING INSTITUTE. 13
Recommendations for Stay Cable Design and Testing.
Design PTI Jan. 1986

PROCEQ SA. 207
Force Measurement on Cable-Stayed Bridges.
Problem Dec.1986.

RAE. 46
Rpt.on a Meallurgical Exam'tion of Sus.Wires from SevBdge...
Test Letter to TRRL, Oct.1978.

RAE. 48
Metallurgical Exam'tion of SevBdge.Hanger CB 27-25U.Pry.Obs.
Test Letter to F.& NP, 27 Mar.1980.

RAE. 50
Metallurgical Examination of Severn Bridge Hanger CB 27-25U.
Test (F)/MT1/16/650/8419/E1140.

RAOOF, HOBBS. 116
The Contrasting Properties of Two Spiral Strands.
Cable CESLIC Report SC4, Aug.1985.

RAOOF, HOBBS. 118
The Bending of Spiral Strand & Armored Cables Close to Term'
Cable,Localised Jour.of Ener.Reso.Tech; V106, Sep.1984

REES, Kim SJ. 144
Jindo and Dolsan Bridges: Construction.
Bridge ICE Proc.Pt.1, Dec.1985.

REHM, PATZAK, NURNBERGER. 92
Cast Metal Anchors for High-Tensile Wire Tendons.
Socket,Cable,Local. Wire, V.26,N.5, Sep/Oct.1977.

RELIANCE ROPE ATTACHMENT Co.Ltd. 51
Test Certificates for Adjustable Socket Assemblies.
Test 24 Oct.1980.

RIGON, THURLIMANN. 165
Fatigue Tests on Post-Tensioned Concrete Beams.
Problem Institut fur Baustatik..Zurich,Aug1985

ROBISON. 130
Cable Stays Catch On.
Design,Bridge Civil Engineering, Jun.1986.

ROMANIV, LAZ'KO, KRYS'KIV. 81
Relationship of Int.Friction to the Fat.Life of Pat.Sl.Wire.
Test Soviet Materials Sci;V19,N6,Nov/Dec'83

ROSSETTI. 205
Design Selection of Wire Ropes Based on Fatigue.
Rope IABSE Reports Vol.37.

SANCHEZ-GALVEZ, ELICES, VALIENTE. 204
Fatigue Crack Propagation in Steel Prestressing Wires.
Test IABSE Reports Vol.37.

SAUL, SVENSSON. 105
Handling of the Loading (Dead Load) of Cantilever Cab.Bdges.
GERMAN. Design Bauingenieur 58(1983).

SCALZI, McGRATH. 76
Mechanical Properties of Structural Cables.
Cable ASCE, V.97,N.ST12, P.8604, Dec.1971.

SCANLAN. 142
Interpretation of Sect.Model Aeroelastic Results in the.....
Wind Univ.of Pittsburgh,Pennsylvania,Oct'85

SEEE INTERNATIONAL. 158
SEEE Prestressing Cable for the Stay of Cable Stayed Bridge.
JAP&ENG.Cable,Bridge FIP Delhi, Feb.1986.

Author
Title
Key-Words Source
SMITH, STONESIFER, SEIBERT. 91
Increased Fat.Life of Wire Rope Through Periodic Overloads.
Rope,Test Offshore Conf.10th Annu.Texas,May 1978
SMITH. 106
Wind Induced Dynamic Response of the Wye Bridge.
Bridge,Wind Engineering Structures,V.2,N.4,Oct1980
STAHLBAU. 171
Die Deutschen Rheinbrucken.
GERMAN. Bridge Stahlbau-Verlag GMBH Koln, 1956.
STAHLBAU. 172
Vorschriften Normen und Profile.
GERMAN. Bridge Stahlbau, 1968.
STAHTLON. 178
Poster on 3 Bridges.
Bridge 3/11/86
STONESIFER, SMITH. 88
Tensile Fatigue in Wire Rope.
Rope Offshore Conf.11th Annu.Texas,May 1979
STRONGHOLD. 115
Cable Stayed Bridges.
Design Publication No.54, Jan.1986.
SURRIDGE. 131
Photos of Luling Bridge, Louisiana.
Bridge K.J.S. Feb.1986.
TAIDO. 39
Yamatogawa Bridge: Design & Constrn.(from C.Eng.in Japan).
Bridge,Wind V.22, 1983,
TAKAHASHI, TABATA, HARA, SHIMADA, OHASHI. 166
Tension Measurement by Microtremor Induced Vibration ....
Problem IHI Eng.Review V.16, N.1, 13 Oct.1986.
TAMHANKAR. 126
Design of Cables and their Anchorages in C-S Girder Bridges.
Cable,Socket,Local. Indian Roads Cong; V.37,N.3, Oct.1976.
TANAKA. 40
Tokyo Bay Shore Road. (from C.Eng.in Japan).
Bridge V.22, 1983,
TANG. 72
Design of Cable-Stayed Girder Bridges.
Design ASCE, V.98,N.ST8, P.9151, Aug.1972.
TANG. 73
Design of Cable-Stayed Girder Bridges.
Design ASCE, V.99,N.ST7, P.9826, Jul.1973.
TAPPIN, CLARK. 143
Jindo and Dolsan Bridges: Design.
Bridge ICE Proc.Pt.1, Dec.1985.
Task Committee of ASCE. 60
Tentative Recommendations for Cable Stayed Bridge Structures
0 Proc. ASCE, N.ST103, May 1977.
TAYLOR, DEMERS. 69
Design,Fabrication & Erection of the Papineau-Leblanc Bridge
Bridge Canadian Struct.Eng.Conference, 1972.
TAYLOR. 65
Cable Stayed Bridges and their Potential in Canada.
Design,Bridge Eng.Journal,Can; V.52,N.11, Nov.1969.
THAON DI REVEL. 110
Research on Endurance of Wire to Fatigue.
Rope,Test Int.Ropeway Rev; V.12,N.1,Jan/Mar 1970

Author
Title
Key-Words Source
TILLY. 32
Design and Performance of Stay Cables.
Design,Bdge,Problem TRRL,Paper toCzech.Sci.&Tech.Soc;Oct86
TINER. 87
Failure Analysis for Wire Rope.
Rope SAMPE Quarterly, V.11,N.1, Oct.1979.
TROITSKY. 34
Cable-Stayed Bridges - Theory & Design 1977.
Book
TRRL. 43
Tests on Apparatus for Measuring Tensions in Hanger Cables.
Test TN781, Jan.1973.
UNIVERSITY COLLEGE. 53
Report on Tension Tests on Cable Clamp Bolts.
Test Cardiff (undated).
UNIVERSITY OF BRISTOL. 55
Results of Tests on Hanger CB 27/25U.
Test 26 Feb.1980.
VINOGRADOV, ATATEKIN. 113
Internal Friction due to Wire Twist in Bent Cable.
Rope,Test Jour.of Eng.Mech; V.112,N.9, Sep.1986.
VIROLA. 179
Principal Bridges of the World.
Bridge 2/25/82
VOEST-ALPINE. 167
Referezliste, Brucken Bridge.
GERMAN & ENG. Bridge Vereinigte Osterreichische Eisen .....
VSL INTERNATIONAL LOSINGER LTD. 18
VSL Stay Cables for Cable-Stayed Bridges.
Design,Bridge Jan. 1984
VSL INTERNATIONAL TEST-REPORT. 198
Summary of Fatigue Tests.
Test, Specification (VSL), 21/11/86.
WARNER, HULSBOS. 162
Fatigue Properties of Prestressing Strand.
Test PCI Journal, Feb.1986, Proc.Paper.
WATERHOUSE, TAYLOR. 101
Fretting Fatigue in Steel Ropes.
Rope,Specification JAS.Lubrication Eng;V.27,N.4, Apr.1971
WILSON, BELL. 145
Design & Constr.of the R.Rother C-S Footbdge,RotherValleyPk.
Bridge ICE Proc.Pt.1, Paper 8842, Jun.1985.
WILSON, BELL. 146
Discuss.Design & Constr.R.Rother C-S Footbdge,RotherValleyPk
Bridge ICE Proc.Pt.1, Paper 8842, Jun.1986.
WYATT. 22
Internal Damping in 38mm Specimens.
Test Sep.1978
XERCAVINS, PERCHERON. 17
Cable Stayed Bridges.
Design,Bridge Freyssinet Magazine No.1 1985

APPENDIX D

APPENDIX D

SAMPLE CALCULATIONS

The notes below serve as a brief example of the use of the design criteria recommended in the Draft Guide Specification for the Fatigue Design of Cables in Cable-Stayed Bridges presented in Appendix E.

Consider a bridge similar to Erskine Bridge in Scotland. This, a single plane, single stay, steel decked cable-stayed bridge with a main span of 1000 feet using cable-stays formed from galvanised spiral bridge strand.

When a single 72 kip HS20 truck is tracked across the structure

(i) the maximum additional cable tension experienced is 150 kips (667 kN)

(ii) the maximum relief of cable tension experienced is 20 kips (89 kN)

Hence the maximum load range in the cable-stay is 170 kips (756 kN).

The cross-sectional area of the cable-stay is 175 sq ins (0.113 sq m)

Hence the maximum axial stress range in the cable due to the passage of a single HS20 truck is approximately 1 ksi (6.7 N/sq.mm).

The structure carries dual two lane carriageway plus cycle tracks. If this is considered as an effective 6 lanes of traffic then full HS20 loading gives a maximum stress range in the cable-stays of 75% of 6 lanes of 1 ksi =
4.5 ksi (31 N/sq mm)

The maximum stress range due to be bending could be as high as 2.5 ksi (17N/sq.mm).

Hence total stress range is 7 ksi (48 N/sq.mm).

Since this is a single stay bridge the cables might be considered as non-redundant.

The type of road fits into Case I of Table 2 of the Draft Guide Specification and hence 2 million cycles shall be considered.

The type of stay used is Galvanised Bridge Strand.

Hence using Table 1 of the Draft Guide Specification it can be seen that the allowable design stress range is 12 ksi.

Hence it can be concluded that the fatigue design of the cable-stays is satisfactory.

APPENDIX E

APPENDIX E

DRAFT GUIDE SPECIFICATION

FOR THE FATIGUE DESIGN OF CABLES IN CABLE-STAYED BRIDGES

CONTENTS

DRAFT GUIDE SPECIFICATION

FOR THE FATIGUE DESIGN OF CABLES IN CABLE-STAYED BRIDGES

1.0 SCOPE

This Guide Specification pertains to the design, manufacture and testing of steel stay cables for cable stayed bridges using round or shaped wire, strand or bars as the main tension element. Requirements are presented for stay cables used in redundant and non-redundant cable-stayed bridges. These requirements are intended to be used in conjunction with the appropriate provisions of the "Standard Spepcifications for Highway Bridges" 13th Edition, The American Association of State Highway and Transportation Officials 1983".

2.0 DEFINITIONS AND NOTATION

2.1 Definitions

Stay - The complete tension member including anchorages, main tension elements, sheathing and all corrosion protection materials, clamping and damping devices and saddles, all anchored into a local area of the superstructure.

Cable - The complete stress carrying metallic section of the main tension components which are anchored with a single socket. This does not include any sheathing, blocking agents, clamps, sockets or anchorage components. (See Fig. 1)

Bundle - A group of open or close packed parallel cables, held together by clamps or wrapping so forming a single stay.

Wire -	The smallest single tension component in all cables except for parallel bar cables. Usually circular in cross section but may be non circular as in locked coil strands. Capable of being coiled for transportation.
Strand -	A small number of wires laid helically around a central straight wire not necessarily in the same direction. Capable of being coiled for transportation.
Spiral bridge strand -	A large number of wires laid helically around a central straight wire. The direction of lay of the helix is usually alternated for each successive layer of wires.

Locked coil strand - (sometimes referred to as locked coil rope)

Similar to a spiral bridge strand except that the outer layers of wires (and possibly some internal layers) are formed with non-circular wires so shaped that they interlock with each other.

Steel Bar -

The smallest single tension component in a parallel bar cable, usually circular in cross-section and usually exceeding 0.5" (13 mm) in diameter. It is supplied in short straight lengths connected with intermediate sockets (usually threaded) and not capable of being coiled for transportation.

Blocking Agent -

The material used to fill (or partially fill) the interstices within and around the cable to keep moistsure out and to provide lubrication. Materials such as oil, grease, wax or cement grout may be used. Some may be introduced at the time of laying up the cable, others may be introduced under pressure after the cable has been completed.

Sheathing - Metal or plastic covering used to enclose the cable and provide permanent protection against corrosion or damage. This may be in the form of a pipe designed to resist pressure during injection of blocking agents.

Tape - A continuous flexible adhesively backed tape of polyvinyl flouride (PVF) or similar material which is wound round a stay or sheath to provide a water, ultra violet (UV) and heat proof barrier.

Cable Clamp - A strap tightened round a bundle of cables at intervals to maintain a defined cross-sectional shape.

Anchorage - A device comprising all components and materials such as sockets, bearing plates, pins, threaded rods etc. required to retain the force in a cable and to transmit this force to the structure of the bridge deck, tower or foundations.

Socket - A permanent enclosure at the end of a cable to enable stress to be transferred from the cable to the rest of the anchorage. Stress transfer between the cable and socket is usually by cast alloy metal, epoxy resin, wedge grips, threaded nuts, button heading or a combination of these. The socket is usually a permanent fixture on the cable but may be adjustable under load relative to the rest of the anchorage.

Wedge Grips - Tapered metal components for transferring the tension by friction from a wire, strand or bar into a socket or other anchorage component.

Button Head - A cold formed upset end to a wire that enables the tension to be transferred to a socket or other anchorage component in bearing.

Socket Filler Material - A material introduced whilst liquid into a socket to surround the individual cable elements and which then solidifies or sets hard to provide a structural bond between cable and socket.

Saddles - A curved trough-shaped structural component that supports one or more cables and permits them to change direction without excessive bearing stresses. Note that saddles may be used to transfer a proportion of cable tension to the structure. In this case the saddle may be considered as a type of anchorage.

Splay Saddle - A saddle which enables individual cables in a bundle to diverge to separate anchorage points. It may nor may not be supported in contact with the structure.

Damper - A device clamped to a stay to absorb vibration energy. May be freely attached intertia type or viscous type fixed between stay and structure, or between stays.

Fatigue life - The endurance expressed as the number of cycles to a defined failure condition under a given constant amplitude stress range.

Fatigue Limit - The constant amplitude stress range below which fatigue life is infinite.

F'_s - The ultimate strength of the steel used in the main tension components of the cable.

The Engineer The person or persons ultimately responsible for the design and/or maintenance.

Design Life Unless otherwise specified, an 80 year life is assumed.

2.2 Notation

Notation is in accordance with the AASHTO Specifications with additional terminology as listed above.

3.0 LOADS, ANALYSIS AND DESIGN CRITERIA

3.1 Loads

The loads causing fatigue effects shall be derived in accordance with AASHTO Div. I Section 3.

Potential aerodynamic effects shall be considered and precautions shall be taken to prevent significant stress fluctuations in the cables from this effect (such as choice of cable profile and provision of adequate damping or restraint). This also applies during the erection period.

Only truck loading shall be used for determining traffic loading fatigue effects.

The impact formula in Clause 3.8.2 of the AASHTO Specification need not be applied for cable stays.

3.2 Analysis

Axial and bending stress ranges in cable-stays due to traffic and wind loading effects shall be added together. In particular, consideration shall be given to bending stresses calculated at the points of rotational restraint in the cable (e.g. anchorages, saddles etc.) arising from fluctuations in axial tension and lateral loading on the cable.

3.3 Design Criteria

3.3.1 Corrosion Resistance

The stay must be designed to ensure long term resistance to penetration of moisture over the design life of the structure. See Clause 7.3 for details of the corrosion resistance acceptance test.

3.3.2 Inspectability

The design of the cable and anchorages shall be selected to provide maximum ease of accesss for inspection for possible deterioration in service

3.3.3 Replacement

The cable-stay system shall be so designed that each cable can be replaced economically without disruption to traffic flow over the bridge.

3.3.4 Damping of Wind Effects

Suitable damping mechanisms shall be provided where necessary to minimise aerodynamic effects on cable-stays.

3.3.5 Design Stresses

The allowable fatigue stress ranges for cable-stays shall be derived from Table 1 below:-

Type of Stay	Design Stress Range ksi			
	Redundant		Non-redundant	
	For 500K Cycles	For 2000K Cycles	For 500K Cycles	For 2000K Cycles
7 wire strand, uncoated (ASTM A416) Uncoated wire (ASTM A421)	25	17	17	14
Galvanized Bridge Strand (ASTM A585)	21	15	15	12

TABLE 1. ALLOWABLE DESIGN STRESS RANGES FOR CABLE STAYS

The number of cycles of stress to be considered shall be obtained from Table 2. below for the appropriate road type.

Stress Cycles*				
Cable Stays in Cable-stayed Bridges				
Type of Road	Case	ADTT**	Truck Loading	Lane Loading
Freeways, Expressways, Major Highways, and Streets	I	900 or more	2,000,000	Not applicable for cable stays
Freeways, Expressways, Major Highways, and Streets	II	less than 900	500,000	
Other Highways and Streets not included in Case I or II	III		500,000	

TABLE 2. STRESS CYCLES

*Note that for cable stay fatigue assessment only, this table supersedes Table 10.3.2A in the AASHTO Specification.

** Average Daily Truck Traffic (one direction)

3.4 <u>Information for Stay Manufacturers</u>

The bridge designers shall provide in the tender documents all the necessary details of the proposed structure and its required performance as necessary for the design and construction of the cable stay.

4.0 <u>MATERIALS</u>

4.1. <u>Tension Elements</u>

4.1.1 Uncoated Wire and Strand

Uncoated wire used in stay cables shall conform to ASTM A421 "Specification for Uncoated Stress Relieved Wire for Prestressed Concrete" Type BA, low relaxation grade.

Uncoated strand used in stay cables shall conform to ASTM A416 "Specification for Uncoated Seven Wire Stress-Relieved Strand for Prestressed Concrete" and shall be weldless, low relaxation grade.

Note that oil, grease or other rust inhibitors are acceptable coatings for wire or strand provided that they are removed where necessary prior to socketting. Therefore, Clause 8.6 of ASTM A421 and Clause 8.4 of ASTM A416 need not apply in this respect.

4.1.2 Zinc Coated Wire and Strand

Zinc coated strand in stay cables shall conform to ASTM A586 "Specification for Zinc-Coated Parallel and Helical Steel Wire Structural Strand". The outer wires shall have Class C coating. Each manufactured length shall be prestretched and the modulus of elasticity recorded. Welds in wires are only permitted prior to drawing.

4.2 Anchorage Components

4.2.1 Sockets

Forged sockets used in stay cables shall conform to ASTM A508, "Anchored and Tempered Vacuum-Treated Carbon and Alloy Steel Forgings for Pressure Vessels" Class 2, including

Supplementary Requirement S5 where the minimum Charpy impact energy shall be 25 ft. lbs.at the minimum specified design temperature of the bridge.

Cast sockets shall conform to the ASTM A486 "Steel Castings for Highway Bridges" Class 90 and where minimum Charpy impact energy shall be 25ft.lbs. at the minimum specified design temperature of the bridge.

4.2.2 Other anchorage components

Where not already covered by existing AASHTO provisions materials used in anchorages shall be as specified by the contract special provisions and/or by the specialist contractor at the time of acceptance testing of the stay cable.

4.3 Sheathing

Material properties and wall thicknesses and joints shall be sufficient to withstand handling, erection, grouting pressures, service stresses and environmental degradation.

4.3.1 Steel Pipe

Black steel pipe sheathing shall conform to the requirements of ASTM A53, Grade B. Sampling and testing procedures shall be in accordance with ASTM A673. The (H) Frequency of heat testing shall be used.

4.3.2 Polyethylene pipe

Polyethylene pipe sheathing shall be black polyethylene plastic pipe conforming to the following requirements.

i) PE pipe sheathing in nominal diameters from 1/2 inch to 3 inches shall meet the requirements of ASTM D3035.

ii) PE pipe sheathing in nominal diameters from 3 inches to 6 inches shall meet the requirements of ASTM D3035 or ASTM F714.

iii) PE pipe sheathing in nominal diameters greater than 6 inches shall meet the requirements of ASTM F714.

iv) All PE pipe sheathing shall conform to the requirements given in Clause 3.4.2 of the "Recommendations for Stay Cable Design, Testing and Installation" published by the Post-Tensioning Institute in February 1990.

4.3.3 Other Materials

Other materials may be used, provided that they satisfy the performance and acceptance test requirements of this Guide Specification.

4.4 Blocking Agents

4.4.1 Organic Based Blocking Agents

Organic based blocking agents shall be free from and impervious to air and moisture. They shall be resistant to attack by atmospheric pollutants and shall not contain any compounds which would react chemically with any of the materials used in the stay component. They shall be resistant to ultraviolet degredation unless contained within an opaque sheath. Their viscosity shall be selected to ensure adequate flow into all voids during construction but to prevent undue flow under hydrostatic pressure during service. They shall be chemically stable and not embrittle with age.

4.4.2 Grout

4.4.2.1 Composition

Grout shall be composed of portland cement, water and possibly approved admixtures. Fire sand may be used as a filler in some circumstances where the stay cable has unusually large void areas

4.4.2.2 Portland Cement

Portland cement shall be ASTM C-150 Type 1, and shall be fresh cement without lumps or other indications of hydration or "pack set". The Contractor shall obtain, from the cement manufacturer, certification in accordance with ASTM C-150 for each truckload delivered to the site.

4.4.2.3 Water

Water shall be potable and shall have chloride, nitrate, sulfide and sulfate contents not greater than 20, 13, 100 and 15 ppm, respectively.

4.4.2.4 Admixtures

An admixture to control bleed at the top of the grout lift shall be used in accordance with the manufacturer's recommendations. The admixture shall not include chloride, nitrate or aluminum powder, and shall be in pre-measured amounts for batch sizes used.

4.5 Other corrosion protection materials

Corrosion protection materials in combination with steel or polyethylene sheathing and grout as defined above may consist of epoxy coating, plastic sheathing around individual wires, strands or bars, spirally wrapped tapes or other membrane materials. Specifications for these materials shall be included in the contract special provisions and/or by the description of the stay cable system provided by the specialist contractor at or before the time of acceptance testing.

5.0 DOCUMENTATION FOR PROPRIETARY DESIGNS

5.1 Detail Drawings

The detail drawings shall show full dimensions, including permitted tolerances, of all components of the stays. The drawings shall also show the dispostion of the components with respect to the bridge superstructure.

5.2 Specification of Materials

Specifications shall be provided for the materials used in all components of the stays, and shall include mechanical, chemical and environmental acceptance testing requirements where these are not already covered by this Guide Specification.

5.3 Construction Procedure Documentation

Procedure documentation defines precisely all those techniques of construction (including maintenance and replacement) which could in any way effect the long term performance of the stay. Where techniques require particular skill and control to ensure their effectiveness, and where this is difficult or very costly to verify by non-destructive examination after completion, they shall be subject to procedure tests to prove the effectiveness of the technique.

5.4 Submission of Documentation

5.4.1 Tender Documentation

The following documentation shall be submitted at the time of tender:-

(i) Detail Drawings and Specifications of materials for all components of the stay.

(ii) Outline procedures (or full procedures, if available) for the manufacture, protection, erection and maintenance of all components of the stays.

(iii) Detailed procedure documentation of previous procedure trials and previous acceptance tests where these have been subject to prior acceptance in accordance with this Guide Specification for similar stays and conditions on previous contracts.

5.4.2 Pre-construction Documentation

The following documentation shall be sumitted for approval before the start of construction of the stays.

(i) Detailed procedures for the manufacture, protection, erection and maintenance of all components of the stays including all matters of quality and inspection. (See Clause 6.1)

(ii) Results of acceptance tests showing compliance with Clause 7.

(iii) Full documentation, independently certified, for procedure trials to demonstrate that the proposed construction procedures can be relied upon to produce satisfactory quality. (See Clause 6.2)

5.4.3 Material Test Certificate

The following documentation shall be submitted for approval before each component is incorporated into the stay:

(i) Material and workmanship quality control test certificates.

(ii) Location of each material batch within each stay or set of stays.

6. PROCEDURES AND PROCEDURE TRIALS

6.1 Procedure Documentation

Written procedures shall be submitted to the Engineer for all operations where special controls are needed for maintaining quality of manufacture, erection and maintenance of the stay in service.

The full written procedures shall detail all steps and checks in the manufacturing processes that are followed during the production of each of the individual elements in the cable stay system. The written procedures shall contain check lists of items and operations with actions and targets that have to be carried out and/or achieved in order to ensure the desired consistent quality of the end product.

In a similar way, the full written procedures shall detail all steps and checks in both the erection processes and the maintenance operations that are required for each of the individual elements in the cable stay system.
When procedure tests have already been carried out for

previous contracts the tests need not be repeated again provided that independently witnessed and certified procedures and test evidence of satisfactory performance is provided by the Contractor and that the procedures have not changed in any way. In this case, the design must also have passed the acceptance tests detailed in Section 7 of this Guide Specification.

6.2 Procedure Trials

Where new procedures are proposed which have not been fully tested and documented, trials shall be performed prior to production, to establish that each procedure is satisfactory in every respect.

Procedure trials shall be carried out in the presence of an independent inspector and shall be fully documented and witnessed.

7. ACCEPTANCE TESTING

The following acceptance tests shall be performed and compliance attained prior to manufacture of bridge stays.

One test can be considered to cover other cable stays with identical design details provided that the overall diameter is within ± 20% of the test cable.

Materials used for acceptance tests shall be close to the minimum specified quality limits where they are likely to affect performance.

7.1 Static test to destruction (axial)

A single test specimen shall include two designed sockets prepared to the approved procedures with a length of cable (including sheath and blocking agent where relevant) not less than 15 feet or 50 diameters between sockets, whichever is greater.

The central section of cable shall be fitted with an extensometer of gauge length not less than 2 feet. Small penetrations shall be made in the sheath to enable the extensiometer to strain with the cable.

The cable shall be loaded five times to 50% of its nominal ultimate load followed by loading to failure. The load

shall be applied at a rate not exceeding 30 ksi per minute. Wire draw, strain and wire breakages shall be recorded.
The acceptance criteria shall be:

a) that the ultimate load is not less than the designer's minimum specified ultimate load.

b) that the elastic modulus is within the designer's specified tolerances.

c) that there is no evidence of wire draw at 80% of ultimate load.

7.2 <u>Fatigue tests in air on bare cable (axial)</u>

Three test specimen lengths shall be prepared as in 7.1 using the approved procedures with designed sockets but without sheath and external blocking agent

The specimens shall be tested under a constant amplitude stress range that is 5 ksi greater than that given in Table 10.3.1A for the relevant stress category at 500,000 cycles for redundant load path structures. (See Section 3.3 of this Guide Specification for the relevant stress category.)

The minimum stress level shall never be less than 3 ksi. The Engineer may stipulate a figure higher than this to match that expected in service.
The loading frequency shall be controlled to ensure that the temperature nowhere exceeds 90^{o}F.

The cable shall be fitted with a sensor to determine wire breakages.

The test shall be continued until failure, or 2,000,000 cycles, whichever is reached first and the number of broken wires shall be recorded at every 100,000 cycles..

The number of broken wires shall not exceed 5% of the total in the cable at 500,000 cycles. No failures shall be accepted for stays using bars.

7.3 <u>Fatigue test in air and salt spray on complete stay system (including sheath, sheath joints and blocking agent) [axial and bending]</u>.

A single test specimen shall be prepared as in 7.1 using the approved procedures with designed sockets, sheath and blocking agent where relevant.
The length between sockets shall be 25 feet or 80 diameters, whichever is greater. For sheath systems with intermediate joints, a joint shall be incorporated 3 feet from each socket mouth.

The cable shall be stressed to 75 ksi axial tension and a rotation of plus and minus 1^{o} imposed between the cable and socket about an axis transverse to the cable at each end. The rotation shall be repeated 2×10^{6} times without loss of tension. The frequency shall not exceed 0.5 Hz nor shall the temperature of the cable exceed $90^{o}F$.

During this period, one socket region and the adjacent 3 feet of cable shall be exposed to a salt fog spray. The other socket and adjacent 3 foot length of cable shall have a water based penetrant dye applied to external surfaces of the cable and socket system during the last hour prior to termination of the test.

After completion of the cyclic test, the cable shall be left under constant tension for 24 hours.
The external surface of the cable, sheath and socket shall be thoroughly cleaned to remove all traces of dye. The axial load shall be removed and the cable and socket system fully dismantled to establish extent of corrosion and dye penetration, if any.

The test shall be accepted under the following conditions:

1. There shall be no visible evidence of corrosion on any wire.

2. For cable-stay systems using ungalvanised wires, there shall be no sign of dye penetration to any part.

3. For cable-stay systems using galvanised wires, dye penetration shall not be accepted within the socket. Outside the socket, penetration shall not have

proceeded beyond the second layer of wires from the outside of the cable.

8. <u>SHOP FABRICATION</u>

Shop fabrication shall proceed only when all detail drawings, specifications and shop procedures have been approved.

All the necessary facilities for inspection, sampling and testing by an independent inspector shall be provided at all reasonable times.

9. <u>PRODUCTION TESTS</u>

9.1 <u>General</u>

Components shall be selected by an independent inspection authority from time to time during the construction process and subjected to the necessary tests to provide proof of conformity with the specifications.

When a production test fails to meet the acceptance levels required by the specifications, two additional samples shall be tested. If either of the additional samples fails to meet the acceptance levels, the batch shall be rejected.

9.2 <u>Wire</u>

Wire shall be tested in its final condition (coated, if relevant) prior to assembly in the stay.

Fatigue Strength:

One 16 ft. long wire sample shall be taken from every 10 tons produced from each heat of steel. All prestressing wires and derived test samples shall be marked in such a manner as to ensure traceability during production, transit, storage and testing. The test specimens shall have a minimum length of 3' between grips. They shall be anchored by the same means as in the stay. The wires shall be tested at an upper stress of 0.45 f'_s and a stress range of 25 ksi greater than the value from category B, for redundant local path structures in Table 10.3.1A for the selected number of test cycles. Wires tested without failure for the selected number of cycles and the related stress range, as specified above, shall be considered acceptable (but note that this is no guarantee of performance of the complete stay cable which shall pass the acceptance tests stipulated in Section 7 of this Guide Specification). In case the first valid test specimen from each sample fails the fatigue test, two additional tests shall be made on test specimens from the same sample. If further failures occur, the wire quantity represented by the sample shall be rejected.

9.3 Strand

Fatigue Strength:

One 16 ft. long strand sample shall be taken from every 10 tons produced from each heat of steel. All prestressing strands and derived test samples shall be marked in such a manner as to ensure traceability during production, transit, storage and testing. The test specimens shall be a minimum length of 3' between grips or 3 lay lengths, whichever is the longer. They shall be anchored by the same means as in the stay. The strand specimens shall be tested at an upper stress of 0.45 f'_s and a stress range 10 ksi greater than the value from Table 10.3.1A, Category B for redundant load path structures, for the selected number of test cycles. Strand tested without failure for the selected number of cycles and the related stress range, as specified above, shall be considered acceptable (but note that this is no guarantee of performance of the complete stay cable which shall pass the acceptance tests stipulated in Section 7 of this Guide Specification). In case the first valid test specimen from each sample fails the fatigue test, two additional tests shall be made on test specimens from the same sample. If further failures occur, the strand quantity represented by that sample shall be rejected.

9.4 Anchorage Components

Anchorage components shall be sampled and tested in accordance with the appropriatse ASTM or other manufacturing specifications of other nationally recognised bodies.

9.5 Completed Socket Terminations

One extra sample termination shall be made for every 100 socket terminations of each cable size. The sample socket shall be sectioned longitudinally on a diameter and transversely 1 inch and 3 inches from the mouth of the socket to verify that adequate filling of the socket and wetting of the wires has been achieved.

9.6 Sheathing

Sheathing shall be sampled and tested in accordance with the appropriate ASTM or manufacturing specifications of other nationally recognised bodies with particular attention being paid to the uniformity, strength and water tightness. In addition, one extra joint shall be made for every 100 to be used, and tested for uniformity strength and water tightness to a standard that is at least as good as the parent sheath material.

9.7 Blocking Agents

Blocking agents shall be sampled and tested for uniformity to their specification by a nationally recognised body.

10.0 SITE ERECTION

Site erection shall proceed only when the fabricated components and site procedures have been approved.

The necessary facilities for access, inspection, sampling and testing by independent inspection shall be provided at all reasonable times.