THE DEVELOPMENT OF TRANSVERSELY STRESS - LAMINATED TIMBER ARCH BRIDGES FOR PEDESTRIAN AND MINOR VEHICLE USE

A thesis submitted in partial fulfilment of the requirements for the degree of Doctor of Philosophy

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CERTIFICATE OF AUTHORSHIP / ORIGINALITY

I, Geoffrey J.H. Freedman, certify that this thesis has not previously been submitted for a degree nor has it been submitted as part of requirements for a degree except as fully acknowledged within the text.

I also certify that the thesis has been written by me. Any help that I have received in my research work and the preparation of the thesis itself has been acknowledged. In addition, I certify that all information sources and literature used are indicated in the thesis.

I certify that during the project five refereed papers were published on various stages of the work. They were all authored by me and my Director of Studies, Professor Abdy Kermani.

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ABSTRACT

Over the last fifty years, the Forestry Commission has developed various low cost bridge designs for its large Estate. These bridges are used for timber lorries and pedestrian access. This thesis describes many of these designs and leads on to the development of stress laminated timber as a modern solution, utilising readily available materials to build sustainable bridges. Timber is the only carbon neutral construction material and its use is, therefore, very appropriate in these times of perceived environmental damage. It also happens to be the product of the Forestry Commission. Recently, the major emphasis of land use on the Estate has moved from production of timber to Public access, for the intended benefit of the nation's health. This has exponentially increased the requirement for low cost footbridges with desirable aesthetic qualities. Timber is the obvious choice but the product available from home grown sources is fast-growing and, consequently, of low structural quality. The development has therefore concentrated on structural methods which could utilise this timber.

Mechanical stress lamination had been pioneered in a few locations around the developed world since 1980, so the Author formed a group of consultants and researchers, called Innovative Timber Engineering in the Countryside, to bring these solutions into UK practice. In the UK, species of timber and climatic conditions in which the structures would be used varied greatly from international practice, so further development was required to assimilate existing solutions and devise new ones. This led to considering shallow arch construction for longer spans, instead of cellular or 'T' beam construction used elsewhere. These constructions either trapped damp air or required LVL which has to be imported into the UK.

Over a three year period, since Autumn 2002, a series of laboratory and field tests were carried out on a variety of stress laminated timber arch bridges in order to develop an understanding of the structural mechanisms involved. Test bridges included a 6m span, a 15m span and four 2.1m spans, all built under laboratory conditions and statically load tested. A 20m span was specifically built, in the forest, to be statically and dynamically

load tested while a number of other commercially built bridges were dynamically tested, both with and without a bonded bitumen topping.

The programme of tests highlighted factors which needed further investigation, so a further four laboratory test bridges were built and loaded to investigate the transfer of load and load capacity under varying conditions. The test bridges all had spans of 6m, with different arch rises. A series of loads was applied at varying lateral tensions and measurements were taken of deflections and lateral spread. The purpose of these tests was to confirm past findings and extend the understanding of the manner in which the load is transmitted through these shallow arch structures.

All of the test load effects on bridges showed good correlation with elastic analysis and indicated an optimum span-to-rise ratio for maximum stiffness. Further, there were a number of other factors e.g. lateral bar tension, which could not be integrated into an analytical model, so an integrated approach was required. For this purpose, a parametric study of a series of arches was undertaken. This study considered a range of different spans, rises and structural arch depths and related these factors to stiffness. To integrate the other factors, a generic semi empirical model was used to model all relating factors to the arch stiffness.

The stress-laminated arch bridges have very good load carrying capacity when compared to structures built from heavier alternative materials. They have a span to structural depth limit of approximately 1 to 100 because more slender structures could resonate under pedestrian loading. The bridges are low cost, sustainable, carbon neutral solutions, using home grown material to produce aesthetically pleasing bridges for an increasing recreational market. The work is a very good foundation for future development and forms a basis for a UK code of practice in mechanical stress lamination.

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ABBREVIATIONS AND DEFINITIONS

AASHTO	American Association of State Highway & Transportation Officials
BRE	Building Research Establishment
DSI	Dywidag System International
DTI	Department for Trade and Industry
E	Modulus of Elasticity
EMC	Equilibrium Moisture Content
FC	Forestry Commission
FNF	Fundamental Natural Frequency
HYS	High Yield Steel
I	Moment of Inertia
LDF	Load Distribution Factor
LVL	Laminated Veneer Lumber
MOR	Modulus of Rupture
QSE	Quality Structural Software for Engineers
RHS	Royal Highland Show
SLS	Serviceability Limit State
SLT	Stress Laminated Timber
SLTA	Stress Laminated Timber Arch
SSSI	Site of Special Scientific Interest
TRADA	Timber Research and Development Association
UK	United Kingdom
ULS	Ultimate Limit State
UTS	University of Technology, Sydney

CHAPTER 1

1 INTRODUCTION

Ancient legend, archaeological studies and a fairly recent article in the Hindustan Times state that the first known man-made bridge apparently dates back 1,750,000 years. Thirty metres long and constructed of a chain of limestone shoals this 'bridge', or causeway, spans the Palk Straits, between India and Sri Lanka and is known as 'Adam's or 'Rama's Bridge' [1]. Whether or not this is believable, it does seem that, since the beginning of time Man has been building bridges, using whatever materials were available, in order to span the gap that those materials would allow. Great innovations have resulted from the diversity of cultures as well as the materials. In the temperate zones, with modest vegetation and many glacial deposits, stone arches were developed, possibly inspired by natural arches formed during geological activity. Simple timber beam bridges may have resulted from fallen trees over small rivers and, in the tropics, vines might have inspired suspension bridges.

As mathematics developed, together with an understanding of materials, the first 'designed' structures would have been built. These would employ joints to extend the spanning capabilities of the timber by creating haunches and forming the first simple trusses. In time, trusses and trestles would be developed to form long span timber bridges. In China 3000 years ago Rainbow arch bridges employed short lengths of straight timbers to form arches built up from corbels joined at mid span. However, joints were difficult to make and maintain and timber bridges had their limitations.

For thousands of years only the basic materials were available and development of long span bridges for heavy loads could not take place. Between one and two hundred years ago steel and concrete became available and new designs were made possible. At the same time, western society was experiencing an industrial revolution which created a demand for strong vehicle bridges. These circumstances were responsible for the development of steel trusses, steel cantilevers and suspension bridges. The new materials were thought to be relatively indestructible and indeed they were, when compared to badly designed timber structures, which rotted quite quickly. For that reason, bridge construction generally ignored timber except for use in rural settings, for short spans, where timber was available. Stone arches were also abandoned because they were labour intensive, and thus, too expensive.

Over the last fifty years, vehicles have more than doubled in weight and bridge design has had to keep pace. At the same time, due to economic restraints, investment in maintenance has reduced in most developed countries. This has led designers to look back through history for ideas on how to create low cost medium span bridge replacement designs which, with some modern engineering could satisfy present day specifications. It was this situation which produced the first mechanical stress laminated flat decks in the USA. The combination of that particular development, together with a personal fondness for the strongest, most durable and 'original' of designs, the stone arch, led the Author to consider developing the first ever stress laminated timber arch bridge. That long love affair with the 'arch', together with forty years of design experience, created the inspiration to combine two technologies from thousands of years apart, to produce an innovative solution suitable for the present time. The offspring of this union is the 'stress laminated timber arch'.

The focus of this PhD Thesis is on research into the load responses of transversely laminated, stress laminated timber (SLT) arch bridge decks. It is an extension of international work on flat and cellular decks.

1.1 Research Objectives and Drivers

Mechanical stress-lamination methods have been successfully used for construction of flat timber bridges for over two decades. The research programme outlined in this thesis details the use of small sections of relatively low grade softwood to design and construct large and attractive bridges. The method utilises the strength properties of timber in an arching action, which contributes significantly to the overall strength and stiffness of the bridges.

This thesis expands the recent work in mechanical stress-lamination of timber by implementing an arching profile for bridges to exploit timber's excellent compressive and end-bearing properties. It details development, assembly and testing of a number of stress-laminated arched timber bridges with spans between 2.1m and 20m. The test results will show that the bridges possess considerable strength and stiffness when subjected to high load levels. Thirty major bridges are now in service as a result of this work. The research programme was designed to explore the full potential of these structures and also to examine the use of relatively low grade UK grown timber species. The influence of different arch profiles and alternative stressing mechanisms were the main focus of the research work.

The aim is to develop decks appropriate to UK conditions in terms of materials available and uses of the bridge decks.

Full and part scale decks were load tested in order to derive a detailed knowledge of global and local effects. The outcomes are an in depth understanding of the structural performance and behaviour of stress laminated arch bridges which can be utilised to analyse footbridges, leading to the future design of vehicle bridges.

The main drivers are:

- an abundance of plantation timber coming on stream in the UK
- a need for many new footbridges created by the recent new access legislation
- a need for many new rural road bridges for forestry
- a need to replace decaying stone arch bridges

1.2 Scope

To develop design rules for the analysis of SLT arch footbridges and provide a basis for the design of similar road bridges. This was done through the application of static and dynamic loads to a number of full scale, long and short bridges and measuring the effects. Testing concentrated on the static load effects within the range of application to realistic practical structures, that is, to determine the load effects on stable timber arch structures.

An optimum range of span-to-rise ratio arches was used to develop generic formulae, thus avoiding slender structures which would fail dynamically, or very upright structures which require spandrel support for strength.

Maintenance of completed structures, particularly the tension bars, has been given detailed consideration.

1.3 Timber as a Construction Material

1.3.1 Species

Timber is the only renewable structural resource available to man. It is carbon neutral because at the end of its useful life the CO_2 which it puts into the atmosphere is equal to the quantity that it removed while growing.

There has been resistance from modern Engineers to the use of timber, partly because of their general lack of understanding of species, properties of strength and durability, and of the construction details necessary for sound construction. Timber frame housing has flourished over recent years but some poor workmanship and detailing has led to a number of premature failures. This has generated new resistance to the use of timber for permanent structures. However, the use of timber for major high profile structures like bridges could help renew confidence.

There are three groups of trees – softwood, temperate hardwood and tropical hardwood [2]. Within these groups there are a number of species in each with different properties. The softwoods generally come from temperate zones and the species which are readily available in the UK are Pine, Spruce, Larch and Douglas Fir. Within these species there are a number of particular tree types. The type which has been planted in the UK over the last eighty years, due to its toleration of the low winter temperatures in Scotland, together with its ability to thrive in the acidic soil resulting from deforestation, is Sitka Spruce. This is very good for paper and pulp but weak as a structural timber, partly because of the rate of growth. The average rate of growth in the UK is approximately forty years to maturity.

Scots Pine is very useful, reasonably abundant and very good at taking up preservative treatment. Larch is more durable in its natural state and takes up less preservative treatment. Douglas Fir is a fine structural timber with good resistance to decay. These are the timbers which will be used for the bridges in this thesis.

Common temperate hardwoods, or deciduous trees, in this part of the world are Oak, Beech, Birch and Lime. These have very variable durability and their 'hard' description is not always deserved. In fact, some 'softwoods' in the world are harder than hardwoods. Oak is particularly useful in the UK as it is plentiful, durable and very hard, though it is very expensive. Beech and Birch are generally used for furniture or other indoor products.

The tropical hardwoods are dense, strong and durable. They are very effective as structural materials and their chemistry gives them resistance against predators, in some cases even marine borers. Greenheart is a famous example and originates from Guyana in South America. In colonial times, this was harvested and used in many UK Victorian maritime piers. The Author built a new pier three years ago using Greenheart which, although it had already given one hundred years' service in Helensburgh pier, was still in perfect condition. These hardwoods however are difficult to work, produce toxic splinters and are difficult to obtain from truly sustainable sources. Their natural habitat tends to be in politically sensitive areas, so their use is best discouraged in preference of plantation softwood.

Plantation softwood is available from most countries in the world so the cost should remain competitive for some time to come. International competition should ensure control of the price, in contrast to that of oil and gas which are only available in a few locations and where the price is controlled politically. This is one very good reason to develop the use of timber for structural applications, even if it is not the strongest, most durable material.

1.3.2 Properties

Timber is not a homogenous material like steel, which has the same properties in all directions. Timber is orthotropic, which means it has different properties in different directions. This is because it is formed from a group of parallel vertical tubes which convey water to the extremities, where sunlight converts it to food through photosynthesis. These tubes form what we call the 'grain' and vary in diameter, depending on growth rate. Along the grain they provide substantial stress resistance but at right angles to the tubes the timber is easily compressed, especially in softwood species. This is useful for construction of SLT bridges because it creates a certain amount of tolerance for timber thicknesses. Hardwood SLT decks can loosen off very easily because they do not compress as much as softwood.

A perfect piece of timber is very strong. Scots Pine can achieve 46N/mm^2 [3] in bending stress before breaking, but natural defects like knots, slope of grain, compression wood etc. mean that a safe stress would only be one tenth of the ultimate.

Fresh timber is made up of about 50% water by volume and considerably more by weight. Most of this water is lost over time after the tree is cut down. Usually it is kiln dried nowadays, to accelerate that process until it has reached the in-service moisture content (MC) which is 16% - 18% for protected external timbers. Even after optimum MC is reached there are seasonable shrinkages, dependent on humidity. Softwood shrinkage is less than temperate hardwoods, which is very satisfactory for SLT bridges.

1.3.3 Durability

This is a description given to the resistance to fungal or insect attack. In this part of the world insect borers are only a problem near seawater and fungus only grows if there is sufficient moisture. Some timbers have natural toxins which resist organic attack while others are very vulnerable. Timber can be impregnated with chemicals to help the resistance to decay. Hardwoods are generally more durable but this is very variable e.g., whilst home grown oak is reasonably resistant, tropical Greenheart is almost totally resistant. Softwoods are all susceptible to decay though Larch, while resistant, still benefits from chemical treatment. Scots Pine or Douglas Fir will be used for SLT bridges because of their strength, availability and resistance to distortion. Scots Pine requires treatment and absorbs it well, whereas Douglas Fir, although moderately resistant, will still be treated.

Treatment of timber is a very emotive environmental subject because the best treatments, designed to kill carbon based insects and fungus, are also toxic to humans. The most superior is Creosote, followed by Copper Chromium Arsenic (CCA) and then Copper Chromium Phosphate (CCP). Organic preservatives can leach out so have a poor performance, externally. Creosote and CCA were legislated out of use in the UK in June 2004 but derogations were introduced for a number of professional uses, like railway sleepers and bridge decks. However, in practice, these treatments have not been available for bridges due to the low demand, so specifications have to employ CCP. Nevertheless, even with treatment, the timber decks will be further protected by a bitumen topping, to ensure water is shed and the deck remains relatively dry. With modern pressure/vacuum treatment processes, the chemical only penetrates a few millimetres so all cuts and holes have to be made before treatment. It is always good practice to protect timber from prolonged wetting therefore, to ensure this, good detailing is essential for timber bridges.

1.3.4 Strength

The strength of a piece of timber depends on its species, growth rate and defects. The growth rate determines the number of growth rings per width of a piece of timber and the closer they are, the stronger the sample. Defects are - clusters of knots, slope of grain and 'wane', which is the loss of edge pieces caused by milling too near the extremities of the round timber. An assessment of all of these factors gives a piece of timber a specific grade which is related to its ultimate strength, that being a proportion of its yield strength or breaking strength. That strength is preceded by a 'C' for coniferous if it is softwood and 'D' for deciduous if it is hardwood. The latest Code of Practice for structural timber design in the UK/Europe is 'Eurocode 5', [51] which is a Limit State Code and uses these Ultimate strength Grades. The old UK Code, 'BS 5268' [50] is still in use and is based on permissible stresses, which are generally about one third of the ultimate. The two common Grades of timber are 'C16' and 'C24', which relate to visual grading categories 'General Structural' and 'Special Structural'. Home grown Oak will be graded as 'D30' or 'D40'. For SLT bridges the main deck timbers will be 'C16' with 'D40' outer oak laminates, to resist lateral bearing pressure from the stressing bars. Although design and analysis will utilise these strength parameters, laminated structures can use higher stresses because the defects of each timber are shared between the laminates. This is certainly true for glue laminated structures but there are some reservations about stress lamination in external environments. However, the evaluation of true modulus and strength grade of a complete deck may become an interesting piece of future work.

1.4 Contents of Thesis

This thesis begins with an overview of timber engineering in the UK and internationally, with a focus on bridges. It looks into the current use of timber and makes particular reference to the Forestry Commission Estate, which has used a number of innovative, low cost, bridging solutions over many years. This has been possible because public and trunk road legislation, which does not permit the use of timber for main structural members, does not apply in this area. The leading chapter finishes by considering recreation structures and other opportunities in the UK for timber structures, but particularly stress laminated bridges. Chapter Two sets out the reasons, and makes a case, for this particular PhD study.

Chapter Three is an historical look at past innovations and good practice in Forestry Commission (FC) bridge design and construction. It is a first-hand account of the development of low cost bridge engineering for rural road networks in the UK. It highlights how materials, engineering expertise and fashion govern the designs of the day and shows why Stress Lamination of Timber is a natural progression for this decade. For the first time, it appears that designs for Estate bridges could be extended for use on minor public roads. This is of particular importance today as Central Government policy has almost removed maintenance budgets for minor public roads and bridges. There is now a movement to convert some minor public roads, from tar surfaced, to gravel roads as in most other parts of the world. Forestry-type bridges would fit well with these new roads and provide significant cost savings.

Chapter Four deals with the history and development of Stress Laminated Timber (SLT) techniques for bridges throughout the world and discusses how that led to the specific development of arches in the UK. The objectives of this research have been restricted to the development of footbridge arches in detail, so that future work will be well founded and taken forward to long-span timber bridges for heavy vehicles.

Chapter Five is a full account of stress lamination knowledge, to date. It takes each aspect of the technique and explains research findings so far, at the various Centres of Excellence in the world. This chapter forms the foundation for the research work of this thesis, by showing the start point for each aspect which required research for the development of the arch structures.

The main body of the research is contained in Chapter Six. It deals with the testing and analysis of a number of arch structures, with the aim of developing design criteria for SLT arch bridge structures. It is a chronological report of the research programme, explaining why each test led to the next. It contains large extracts from published papers which reported the research as it progressed. This contains some of the results, as this was necessary to explain the experimental processes. However, the following chapter is reserved for the full report of the results.

Chapter Seven is the complete report of the data from all test bridges, with a view to concluding the overall findings. At the beginning of the research, comparisons were made between timber arches and those made from other materials. The timber arches were considered to be good in compression with significant bending capacity. It was therefore deduced that the very shallow arches may act as a combination of a beam and an arch, so tests were designed to investigate this. The data showed good correlation between elastic analysis and tests.

Although this thesis indicates that reliable design can be achieved by using elastic analysis, there are other factors which cannot be taken into account by such analysis, the most significant factor being the lateral tension holding the laminates together. This tension creates the friction necessary for load transfer and it reduces, with relaxation of the steel bars and changes in moisture content of the timber. Chapter Eight extends the test results by adding parametric data so that these other factors can be taken into account. This is done by equating each parameter affecting the strength of the arches to a common factor – stiffness. From these relationships a generic formula is derived for the assessment of stiffness for a working envelope of arch shapes and sizes.

The overall objective of this project is to provide the Construction Industry with a simplified design for foot bridges and vehicle bridges which may be transferable to public road use. This objective was inspired by a Department of Trade & Industry (DTI) International Technology Mission [4] in 2000, led by the Author, which toured Scandinavia to look at their solutions. Chapter Nine contains the conclusions from this study which could help towards that goal. It goes on to point out the future work necessary to complete this design technique and take the work forward to new goals.

CHAPTER 2

2 BACKGROUND AND FRAMEWORK TO PROJECT

2.1 Background History of Forestry Commission Estate

The Forestry Commission (FC) holds and manages the largest land area estate in the UK, on behalf of the General Public. It carries out commercial forestry but, very importantly, it provides recreation and access facilities for the Public.

"The mission [5] of the Forestry Commission is to protect and expand Britain's forests and woodlands and increase their value to society and the environment."

In this enlightened time the public landholding is primarily used to improve the health of the Nation and demonstrate environmental excellence. Commercial forestry creates approximately half of the funding for these activities (£82m in the year 2005) but the organisation depends on Government grants to balance the books (£60m). The most vigorous development today is in providing all ability access to the Estate and constructing recreation facilities. At the same time, the commercial arm demands much attention in maintaining the access networks for harvesting the timber resource and for replanting.

These activities require a large holding of roads, Figure 2.1, and tracks and paths which, in turn, have many bridges of all types. Through its eighty seven years of existence, many innovative designs of bridges have been built. Their design has been governed by availability of materials, loading and, of course, cost. These parameters have not changed but through time, the materials available and type of use have changed significantly.

Fig 2.1 - Forestry Commission road

The FC estate is approximately 1 million hectares [6] with about 2,500 significant bridges. These bridges are rural, low cost designs aimed at maximising value for money. The 1500 (approx) road bridges are designed to take the largest lorries permitted on the public roads, as these vehicles need to enter the forest to load the cut timber.

The construction of 44 tonne capacity bridges at a fraction of the cost of similar bridges on the public highways has required some compromise and much innovation, without reducing safety standards. Further pressure to innovate has come from the Public's recent obsession with the outdoors and Society's current interest in maintaining a healthy environment. The recently granted access rights [16] [17] have fuelled the development of many novel activities such as mountain biking and orienteering, which have increased the requirement for sustainable footbridges. Heavily used mountain-bike trails cause ground erosion, so timber paths and track sections are commonly built as a protective measure. These are becoming sophisticated features in order to add greater excitement and all are built from timber. New research into using timber for bridges and other rural structures is therefore in line with market forces.

2.2 International Development of Timber Bridges

2.2.1 History

In the developed world, small span bridges were generally built using timber until a little over one hundred years ago, when steel became plentiful. This was then followed by reinforced concrete. These new materials had the important advantages of lasting longer and of not rotting. In the meantime, many timber bridges were still giving good service but there was a lack of maintenance and a shortage of engineering expertise in timber design to repair them or to build new ones. About twenty five years ago a new interest began in timber, fuelled by the necessity to replace so many of these rural bridges throughout the world, made necessary by a lack of investment in the minor rural road network. There was a need, not only to replace and repair the old timber structures but, in fact, also many of the steel ones, which had corroded. 'Spalling' also became a common fault on concrete bridges, so action was required.

2.2.2 Available Materials and Innovation

During the 19th century and before, there had been a plentiful supply, internationally, of good quality large section timber but, for a variety of reasons, this was no longer the case in the latter part of the twentieth century. Innovation was required to make up for the deficiencies of the small sized, fast growing plantation timber. Glue lamination was an obvious choice, to produce large sections from small timbers, but that required sophisticated precision engineering which increased costs. Stress lamination was explored by the Canadians in the late 1970s and the USA took up the development during the 1980s. It was then that Europe and Australia, who both had the same needs for many rural bridges, began to consider stress lamination. Soon there was an international collaboration and by the year 2000 there were sophisticated designs in many locations, using all forms of composite laminations, in attempts to increase spans.

2.2.3 Timber Bridges around the World

The UK had not been involved in a revival of timber engineering in bridges, except for a few very specialist structures, built more for aesthetic reasons than out of necessity.

Many UK rural bridges are, in fact, stone arches and they have survived the centuries well. However large vehicles used for modern agriculture and forestry were beginning to cause problems in the UK, and similar problems were identified in other countries. It is interesting to note that the USA has 577,000 bridges of which 42,000 are timber and over 100,000 require to be replaced or repaired [7]. In Australia, there are 10,000 timber bridges on the eastern coast alone [*Lembke - 1991*] [8], mostly over seventy years old and also needing to be replaced or repaired. As the UK became industrialised before other countries, and consequently had fewer trees left, we now tend to have old steel, concrete and masonry bridges requiring repair or replacement.

2.2.4 Timber Design Innovation in the UK

Innovative Timber Engineering in the Countryside (InTEC) [9] was formed in the year 2000 to help Timber Engineering develop in the UK. Its first task was to look at developments internationally. A full report was produced in the year 2000 about these developments with a view to Engineers in the UK forming partnerships or collaborating with others working in similar areas of Timber Engineering, in other countries. This marked the beginning of some sophisticated timber bridge engineering in the UK. The participating members are BRE [10], TRADA [11] and Forestry Civil Engineering [12]. They are maintaining international links and bringing new ideas to the UK.

Although this group emerged from the three large organizations involved in Timber Engineering, it is now encouraging universities and smaller groups of Engineers to develop and share their ideas. It provides a free forum for all Timber Engineers and hopes to revive the discipline in the UK, through contact with international Timber Engineers.

2.3 Timber Engineering in the UK

2.3.1 The UK Timber Industry

The timber industry in the UK consists of importers and forestry companies who are the materials suppliers, sawmills who provide basic processing and finally, the retail sector.

The UK produces 8.6 million cubic metres [6] of timber per year and imports 52 million cubic metres of wood pulp, paper etc. The structural quality of the imported timber is generally higher than home grown timber, mainly Sitka Spruce. Because of its faster growth rate and resulting lower structural quality, most home-grown softwood timber is used for pulp and paper or in the construction industry for low grade structural uses. In the UK, there are very few specialist timber designers, only a small number of training establishments and a limited supply of specialist processors who can carry out fabrication. As a result of this, Timber Engineering here is not big business.

2.3.2 Markets

There are, however, large markets with excellent opportunities for timber to replace steel, concrete or plastic. Sustainability is becoming the key to acceptable development in the UK and timber has the credentials to fulfil many of the criteria. House building is set to increase substantially and thousands of new and replacement bridges are required.

2.3.3 Timber Engineering Designers

Building Research Establishment (BRE) [10], Timber Research and Development Agency (TRADA) [11] and a few consultants e.g. Buro Happold and Arup, have worked over the years to maintain some momentum in specialist Timber Engineering but without Institutional involvement, there could never be fundamental research and training. Fortunately Scottish Enterprise made Forestry one of eight Cluster Industries in Scotland [13] and provided funds to set up the Centre for Timber Engineering (CTE) at Napier University [14] in 2002. The aims of CTE include training a new generation of Timber Engineers and carrying out research which will lead to innovative engineering uses for timber. Work here is already producing results and the hope is that it will grow to become a Centre of Excellence, to provide a developing industry with much needed Academic backup.

2.3.4 Timber Resource

Although most of the UK's timber is imported, there are good reasons to use home grown timber for high value added products, where there is more profit to be made. A full life cycle analysis would include the CO_2 emissions from transport of timber, thereby reducing the sustainability of imported material. Presently, most home grown softwood is pulped or chipped so, in order to develop high value uses for fast growing Sitka Spruce and Scots Pine, they must be used as whole wood products. To do this successfully, their weakest properties need to be designed out. Timber must be treated with preservative and reliance on bending strength needs to be minimised. Useful developments have recently taken place with timber cladding and wall studding for timber frame houses, but Public acceptance must be enhanced. A World in Action [15] television programme, ten years ago, discredited timber frame houses by highlighting the tragic results of poor workmanship. It is thought that one way to increase Public confidence could be to build timber bridges on public roads and display what this 'secondary' structural material is potentially capable of.

2.3.5 Timber Lamination

That thought process led to lamination as being the only way to produce large, strong sections from small timbers. However the UK does not have a glue lamination industry or any Laminated Veneer Lumber (LVL) factories. These products need large markets, require enormous investment and only utilise stable, dry, high quality timber along with toxic glues. These facts led to consideration of the potential for Mechanically Stressed Laminated Timber (SLT) with high yield steel stressing bars to provide lateral tension. A search of international practice showed that research and development was advanced and that the technique was successful and of interest for development in the UK. However, it was noted that the span capabilities of slab decks were limited and the cellular decks used in other countries would not be suitable for the damp conditions in the UK. Specific research would therefore be required, at least to adapt existing technology and to develop some new ideas. After some deliberation it was decided that SLT arch construction might be a way of overcoming the long span problem in a wet climate, whilst utilising the compressive strength of timber.

2.3.6 Mechanical Stress Lamination in the UK

The development of stress laminated timber decks in the UK is still quite modest but as research results are published in the UK, interest is expanding. There are about thirty permanent bridges around the UK, designed by the Author, who also supervised their construction. Many more are planned for the coming years. At this time, development is being spearheaded by Napier University with some Forestry Commission funding, under the timber research initiative known as InTEC [9].

The main areas of SLT development will involve:

- which home grown species should be used
- treatment of the timbers
- deciding on a cost effective stressing system
- timber lengths
- position of stressing bars
- maintenance
- foundations
- long spans suitable for UK climate.

The construction of these bridges has been hampered by the lack of a network of timber fabricators and stress graders. The UK industry is set up to produce treated sawn timber for processing on site, except for the few timber framed housing specialists. There are very few companies who will stress grade, cut and drill accurately, then treat timbers for special structures. This facility is easily available in Europe and hopefully will develop as the market grows in the UK. This is where Forestry Cluster industries can help.

2.3.7 Other Timber Research in the UK

Most research in the UK is industry driven and is aligned to the biggest market, which is at present, housing. Much effort has gone into the development of Structural Insulated Panels (SIP), which are Orientated Strand Boards (OSB) with insulating material sandwiched between. These panels will lead to low cost, well-insulated homes and industrial sheds but Public acceptance will still take time. Presently there is also work being done on flitch beams, OSB webbed 'I' beams, timber cladding etc. and all research should help to build Public confidence in timber as a primary structural material.

2.4 UK Bridge & Recreational Requirements and Opportunities

2.4.1 Current Access to the Countryside

England has approximately 190,000 km (118,000 miles) of footpaths, bridleways, by-ways and other rights of way but the Public were not permitted to stray from the paths until October 2005, when new legislation [16] allowed them restricted access to private land. These routes are the most important way for visitors to enjoy the countryside, and are also useful for local people to access shops, schools and workplaces. Highway authorities and landowners have a duty to maintain bridges over natural water courses, including farm ditches, to provide access for the Public. Due to pressures on capital expenditure over the last fifteen years, many bridges have not been maintained and there is now a backlog of sub-standard bridges. Added to that, access authorities (National Park authorities and the local highway authorities) have powers under the Countryside and Rights of Way Act 2000, [16], to make agreements with the owners and occupiers to improve routes on to newly mapped open access land. This will inevitably lead to the requirement for more, new structures. Scotland has its own independent legislation, [17] but the general theme remains the same – to increase safe access to land for the General Public.

2.4.2 New Footbridge Requirements

The number of new or replacement footbridges and small vehicle bridges required in the UK at present is unknown but a conservative estimate would be at least two thousand. Such a capital outlay is not readily available but there are many funding agencies, meaning that simple low cost bridges, which could be assembled by volunteer labour, could find a ready market.

2.4.3 Funders

The Office of the Deputy Prime Minister (ODPM) [84] provides funds to encourage the Public into the countryside in an effort to improve health. This has been used extensively in recent years to fund recreation bridges as well as other facilities which can be built from timber. Lottery funding is also available as well as local enterprise funding and some from private companies but direct Local Authority funding is most common. In Scotland, most of the new bridges for access are being financed by Local Authorities and Enterprise groups, as opposed to land owners.

2.4.4 Funder Specification

Because of the ODPM initiatives and the new access legislation, which also applies in a similar way in Scotland, there are many new opportunities for timber structures in the countryside. Timber fits well with the initiatives which go hand in hand with environmental and sustainability issues. People seem to like timber, especially in the countryside and funders are happy because timber structures tend to be lower in terms of cost. Home grown timber will always be the preferred option as it does not incur large transport costs and is, therefore, more sustainable. However, it is generally of small section and poor in bending stress, making stress lamination one way of overcoming these problems.

2.4.5 Timber Activity Structures

Interesting and exciting activities are valuable in attracting people and encouraging them to take exercise. Walking is still by far the most popular activity but the young are being attracted in very large numbers by mountain biking. This results in severe ground erosion so timber structures are often used to keep the riders off the ground, where soils are delicate or wet. This is leading to a significant number of new structures throughout the UK and is a very good opportunity for the utilisation of stress laminated timber. Viewing platforms and lookout towers are also in great demand so that the Public can gain maximum value from viewpoints in the countryside. A recent development is the creation of timber sculptures and interesting artefacts in the countryside. These exhibits range from specialist timber forest furniture to tree-hanging mirrors, but all include the innovative use of timber. Figures 2.2 and 2.3 show a timber shelter and viewing platform.

Fig 2.2 – Timber shelter - Glenurquhart

Fig 2.3 – Viewing platform – Glenashdale Falls, Arran

2.4.6 Timber Environmental Structures

Wetlands are a main target in recent government initiatives. Millions of pounds are being spent on improving habitats and raising water levels in order to provide catchment storage. These areas become high quality recreation facilities and produce yet another market for timber structures in the shape of boardwalks. Medieval and archaeological sites are also of great interest but again the Public needs to be channelled away from important relics, to prevent the risk of damage to them. Many boardwalks and bridges are required to serve this purpose, Figure 2.4.

Fig 2.4 - Boardwalk at Salcey

2.4.7 Specialist Recreation Structures

Treetop walkways are becoming very popular, enabling the Public to gain a 'bird's eye' view of the forest from unusual and exciting vantage points, Figure 2.5. These are complex structures but again, stress lamination can fulfill many of the requirements.

Fig 2.5 - Aerial walkway Salcey

2.4.8 Private Recreation

Wood for Good [18] launched a campaign in the year 2000 to increase the General Public's desire to have wood in and around their homes. In only five years they have increased the public's interest from 18% to 40% and the patio decking industry is now a major part of the UK's 700,000m^3 increase in annual wood consumption.

2.4.9 New Road Bridge Requirements

Although the greatest market in the UK apart from housing is recreation, timber road bridges for 44 tonne vehicles could increase public confidence in timber as a primary structural material. That makes this use of the material especially important. At present there are very few bridges of this type because of the small sizes of timbers available. However, Stress Lamination can overcome the problems of dimension with multi-member structures. Cellular boxes are not considered to be a good idea in the UK because they would trap damp air and consequently rot. 'T' beam structures require Laminated Veneer Lumber (LVL) beams which are unavailable in the UK, and importing them would not be cost effective. However, SLT arches supporting SLT flat slab decks will work. SLT space-frame decks will also increase spans by increasing inertia while allowing a drying wind through the structure. This is seen as further potential work extending from this research.

CHAPTER 3

3 FORESTRY COMMISSION BRIDGES

Low Cost Bridges and Large Culverts on Forestry Commission Estate - Minor Public Roads and Public Rights of Way bridges

3.1 History of the Forestry Commission Estate

The FC was formed in 1919 by taking over large estates in the UK and planting suitable trees for the, then, perceived markets. Many of these estates were on poor damp ground and Sitka Spruce, from Vancouver, was chosen as the most likely species to thrive. The estates had stone arch and steel beam bridges with gravel roads, which were suitable for the tree planting traffic. However, after the Second World War, stronger roads and bridges were needed for the first extraction lorries, which had increased in size to 20 tonnes. One of the most innovative ideas implemented was a design using old tram-rails, which were being removed from major town centres all around the country. Six hundred bridges were built by placing the ‘T’ rail sections side by side and filling the gaps with concrete, Figure 3.1. Five hundred of those bridges are still in service, fifty years on. Many other innovative designs were tried but none as successful and low cost as the tramrail bridges. Much innovation has also been developed in building low cost unsurfaced roads.

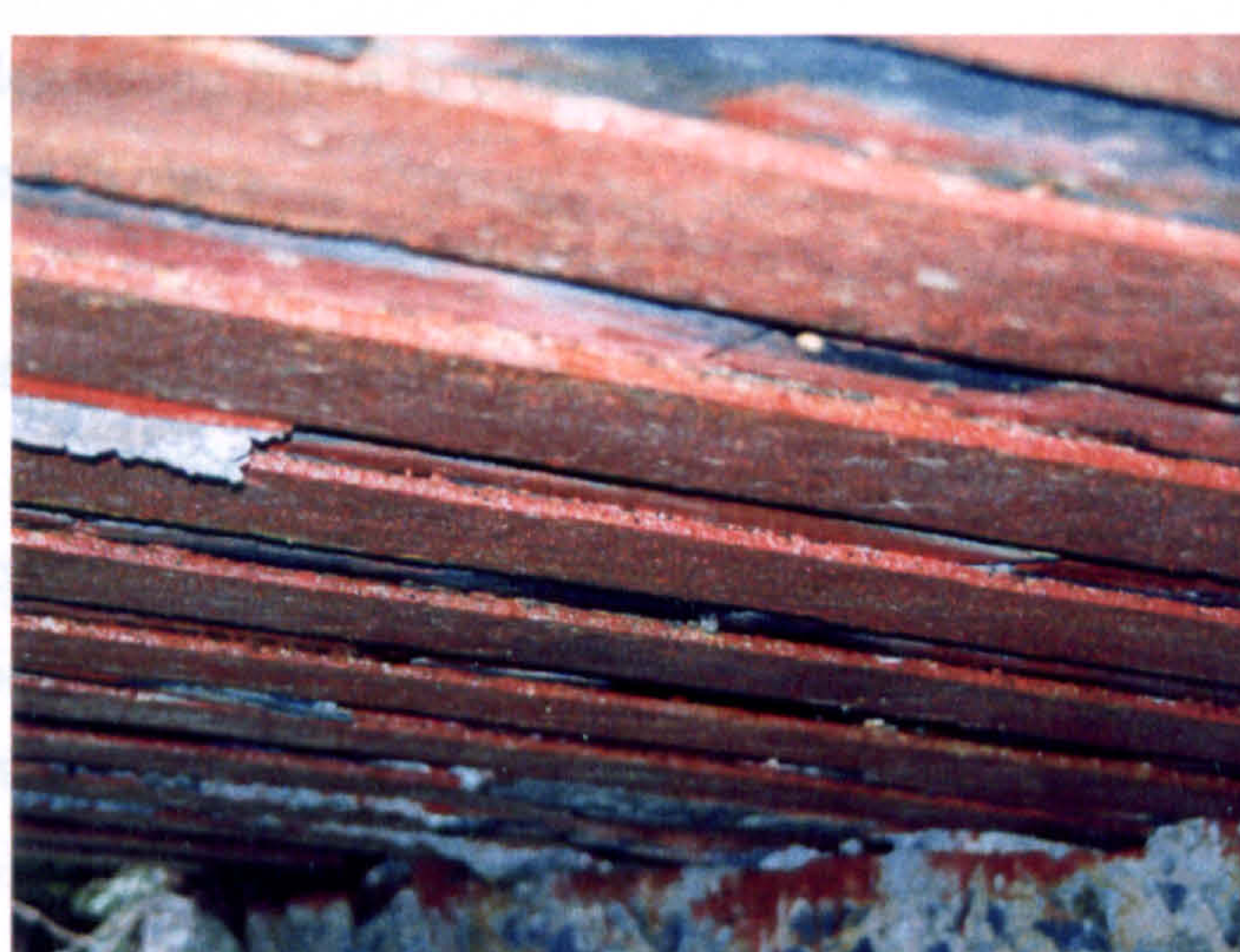


Fig 3.1 - Underside of tramrail bridge

Footbridges were usually rough structures, built by Estate staff that had little or no engineering knowledge. They would often be made by using round timber logs as beams with deck boards nailed directly to the beams. This would frequently cause a split on the top of the beam, into which water would seep, resulting in undetectable internal rot in the heartwood and sudden failure. There were, however, some good examples and also the occasional fine suspension bridge from the Victorian era. One particularly interesting innovation was the 'bucket bridge' which is a one-person bridge, the person requiring to be very strong in order to pull against the catenary, Figure 3.2.

Fig 3.2 – Bucket bridge over the River Spey

It was not until legislation began to control safety that a new approach to the design of footbridges was put in place.

3.2 Introduction to Forestry Bridges

Forestry bridges are required to take all the loads which are legally imposed on public roads but some compromises in design can be made because of slower speeds and low usage, without increasing risk. These factors reduce costs significantly. There has been a debate for some time on the proposal that similar bridges could be used on minor public roads, especially as there is now a need to replace quite a few on the UK's failing network. Further, there is a good economic case for converting some of the minor public roads themselves to a forestry road specification [9].

Footbridge design was given some guidance from British Standards but it was geared to urban design where durability against vandalism is a major factor. Lower, more realistic, loadings were defined in 1984 in a Countryside Commission Handbook [19], about to be replaced by a similar book 'Path Bridges' [20], and are presently used by the FC. Footbridge construction in the UK has grown exponentially over recent years at the same time as extra responsibilities have been placed on landowners for the safety of the Public. This has resulted in accurate Engineering design which can be certified, in order that insurance policies can protect owners. The FC standard designs are used by many public authorities and estates throughout the UK, so there is an extra responsibility to achieve durable, safe, economic and aesthetically pleasing structures.

References will be made to a number of reports and field visits around the world, by the Author, concerning rural bridges. They are described as background to this research in a review in Appendix 6. This text will concentrate on crossings spanning between 2m and 15m because they are the most usual on public roads and the focus of this research programme is to produce design guidance for low cost timber bridges, suitable for pedestrians and, eventually, heavy vehicles.

Many of the current rural road bridges are, in fact, masonry arches. They are usually very strong but of rigid construction and therefore susceptible to frequent heavy loads. They were designed for horses and carts but are also able to take intermittent heavy loading, if given time to 'settle' between loadings. An assessment of these structures by the MEXI method [21], approved by the Department of Transport, can often show that the bridge is strong enough for the loads although there maybe concerns over durability. This becomes a major issue when frequent timber harvesting traffic travels over these old rigid structures which have only experienced, on average, one heavy load per month during their lifetime. Often they are retained for light traffic only and a bypass for heavy vehicles is provided. This is where the forestry specification is well suited. Figure 3.3 shows a recent example of this designed by the author at Brenchoille in Argyll.

Fig 3.3 - Substitute steel and timber road bridge Brenchoille, Argyll

3.2.1 Forestry Commission Bridge Designs Standards

Bridges are provided for pedestrians, livestock, horses, cyclists, light vehicles, Construction and Use traffic [22] or special vehicles. They have abutments, wing-walls, decks and parapets. They need to permit the passage of a 1 in 100 year flood without causing damage to the bridge or the flood plain. They also need to look appropriate in their environment and their cost must be within the budget. A good design will take all of these factors into account and provide the optimum solution within the constraints and guidance given in relevant legislation and codes of practice. Above all, the materials must come from a sustainable source, environmental impact must be minimized and Life Cycle cost must be optimized.

3.3 Vehicle Bridges

3.3.1 Deck Specification

Design generally follows BS 5400 [23] and the relevant Department of Transport Memoranda but savings are made where possible, usually resulting in a slight compromise of the official codes.

3.3.2 Deck Specification - Departures from Standards

The design of forestry bridges can legitimately depart from highway codes of practice because these are generally upland bridges situated near the river sources, resulting in smaller catchment areas. This usually means that bridge decks are relatively close to the river beds, so reducing the consequences if a vehicle were to drive over the edge. The speeds are restricted because of the loose road surface and tighter geometry so there is reduced risk of colliding with a bridge parapet. Drivers tend to be more cautious on gravel roads and in fact there are statistically fewer reportable accidents in Sweden, where most public roads have gravel surfaces [4]. For these reasons any sort of containment barrier is considered unnecessary and a sturdy pedestrian parapet is usually substituted to help drivers align themselves to the bridge span and also to protect walkers. This modification means that edge stiffness to support a parapet becomes unnecessary and allows the bridge to act like a single beam, resulting in better lateral load dispersion and, most importantly, a considerable cost saving.

3.3.3 Deck Specification - Fatigue Loads

The lower number of vehicles using the bridges will mean that durability against constant dynamic loadings can be reduced and fatigue from stress reversals is negligible. These factors allow much design flexibility. For example, beams can be spliced at mid span and bearings can be reduced to a sheet of bitumen felt.

3.3.4 Deck Specification - Timber in Highway Bridges

Timber is used for forestry bridges as a primary structural element. Timber is excluded from British Standard BS 5400 – Highway Structures [23]. However the new Eurocode 5 part 2 [51] deals with timber design for bridges, so it may ease the path for the introduction of timber bridges on public roads. It is understood that the current BS ignores timber because the joints can become loose with age, through wear and rot, and are considered to be below the required durability standards. Many new advances in jointing now supersede that argument and timber is an abundant low cost material, well suited to highway structures. Most other countries around the world permit timber for highway bridges and recent discussions with the Department of Transport suggest that a well designed timber bridge will be permitted in the UK. Figure 3.4 shows one example of many timber bridges used on public roads in Norway.

Fig 3.4 - Timber truss and SLT deck bridge over motorway near Oslo, Norway

3.3.5 Deck Specification - Deck Waterproofing

Salt is not used on forest roads in the winter, so waterproofing of concrete deck structures is not necessary and timber decks do not need protection. This simplifies construction, saving capital and avoiding costly maintenance.

3.3.6 Deck Specification - Simple Spans

Square spans are usually specified because they save money in construction and avoid unequal lateral forces which twist the deck. It is usually more expensive to build a shorter span skew than a longer square span because of the special details and the inevitable mistakes and bad fits. It always pays to keep structures simple and thus have a wider choice of contractors who can construct competitively.

3.3.7 Deck Specification - Deck Width

The decks should be 3.5m wide. This is the optimum to permit large specialist vehicles without allowing two way traffic and keeping the structural costs down. Beams are generally concentrated beneath the wheel tracks of the heaviest loads and the narrow width ensures they cannot stray from that line. Large wheel loads between beams require bigger, more expensive deck boards.

3.3.8 Deck Specification - Deck Weight

Deck structures are designed to be as light as possible. This is a particular advantage when foundations are on poor ground. A steel beam and timber deck is light but the solid timber stress laminated deck could be even lighter. Steel beam lightweight decks are of modular construction and can be dismantled easily. This is an advantage if abutments need to be repaired and they are often the best solution for re-decking onto existing abutments of less than certain ability.

3.3.9 Deck Specification - Loading

The loads cannot be changed from those specified for the public road system. This means that HA [24] (The Department for Transport statistically derived loading for the design of public highway bridges) is adopted but some saving is made by applying the minimum number of HB [24] units. HA is a statistically derived load which allows for all vehicles permitted on the public roads without special permission and takes account of all additional features e.g. dynamic, overload, bunching. These are generally relevant but there is a little spare capacity because e.g. bunching can never become an issue on a single lane bridge. HB is a specialist oversize vehicle not normally allowed for on minor public roads. The minimum allowance, 25 units of HB, is designed for as it does not have a greater effect on short spans than HA. Spans of less than 10m are not usually adversely affected by wind loads. Footbridges, on the other hand, are designed for wind loads [25].

3.4 Vehicle Bridge Types

Two types of deck and beam structure have become the FC standards over recent years and they are suitable for use on minor public roads if the Local Authority agrees with the derogations built into the design:-

Pre-stressed Concrete

Steel Beams and Timber Deck

3.4.1 Pre-stressed Concrete

These bridges are based on a long established design devised by the Cement and Concrete Association in the 1950s. The deck is made from inverted pre-stressed "T" beams produced in a pre-casting factory. They are laid on abutments side by side and in situ concrete is poured in the voids between the beams and over the top of the webs to form a solid concrete mass. The Forestry Commission has built about three hundred of these bridges and they have been trouble free for between thirty and forty years.

This is probably due to the virtually crack free soffit, which is a result of the pre-stress in the beams keeping the underside of the deck in compression. When a heavy load passes this goes into tension but it happens so seldom and statistically, when it does, it is likely to be when the air is dry so no moisture enters the structure and deterioration is therefore minimised.

These bridge decks are heavy, necessitating excellent abutments and, therefore, good ground. They require good access and preferably a reliable source of ready-mixed concrete. A highly skilled construction workforce is necessary to fix the reinforcement steel and to place the concrete as shown in Figure 3.5. This is an excellent solution but there are a few limitations. In the past, the kerb was factory cast in-situ with the outer beams, greatly simplifying the site works but the costs of this luxury have spiraled out of control. Site cast kerbs are very expensive as new work regulations demand a full scaffold on site to cast the kerb. Other forms of kerb beam are being tried, in attempts to keep costs down. However, when these bridges are complete, bare concrete is always visible in elevation, which clients regard as ugly in rural areas. They are also difficult to fix parapets to, which is another disadvantage.

The costs have, in the past, been extremely competitive but they rely on a low cost supply of beams. These are no longer manufactured in Scotland, so transport costs have become a more significant element.

These may well be the first choice for a minor public road bridge because of their durability but e.g. as a bypass bridge perhaps a less permanent solution, which could be dismantled easily, would be more appropriate in some circumstances. As with most design briefs, each case will have different conditions and a best solution.

Fig 3.5 - Pre-stressed concrete bridge - Stirling

3.4.2 Steel Beam and Timber Deck

This is a design introduced to the Forestry Commission about twenty years ago and one which deviates from the BS codes more than the pre-stressed concrete deck. It is based on a design used for Estate bridges one hundred years ago, when steel beams became available. In those early bridges the steel corroded badly and the timber was not pressure-treated so they deteriorated quickly and gained a bad reputation. The new design utilises modern materials and processes and has proved to be the first choice of bridge in the span range to 15m for the past fifteen years.

The beams are standard hot rolled high tensile grade Universal Beam steel sections which are galvanised immediately after fabrication. This ensures a maintenance free life of fifty years in the rural environment. They are arranged in two pairs, forming two sets of "rails" across the span. Timber runners are bolted to the top flanges and transverse timber deck boards are bolted to them. Where high durability is required, longitudinally tensioned SLT deck sections should be used directly bolted to the top flanges of the beams. Bracing, on spans up to 10m, is provided by diagonal timbers bolted through to the deck timbers. On spans from 10m to 15m, steel channel diaphragms are provided at 4m centres and 15m to 20m diagonal steel channel bracing. It is a simple solution but great care is required during construction to ensure the necessary durability. Regular maintenance is required to ensure in-service performance.

About fifty similar bridges have been built over recent years and only two have given any trouble, both with the same problem - the deck boards became loose. In one case, 500,000 tonnes of timber travelled over the bridge in a short space of time and tracked vehicles also used it. This caused some loosening of the deck boards and as immediate maintenance was not carried out, progressive loosening resulted. In the other case, the road alignment was poor and large vehicle rear bogies scrubbed across the surface, loosening the boards. In each case the remedy was to use larger coach screws and deck runner boards. The design is now well tested and there is confidence that those bridges will give good service for at least their fifty-year life, providing that the deck boards are maintained and changed every fifteen years.

The main advantages of the steel beam timber deck design are:

- ease of construction
- can be built in any weather conditions
- lightweight on the abutments
- can tolerate some differential settlement
- ease of fixing parapets
- fits well in the forest environment
- low cost.

Figure 3.6 shows two examples of steel beam and timber deck bridges in a forest environment.

This design had been used in a public road situation where a masonry arch could not take forestry traffic. Instead of replacing the existing bridge, it was retained for restricted traffic and a bypass bridge was built for heavy vehicles. This was part of a major scheme at Brenchoille in Argyll where the public road was ripped up and graded to form a forest type road, Figure 3.3.

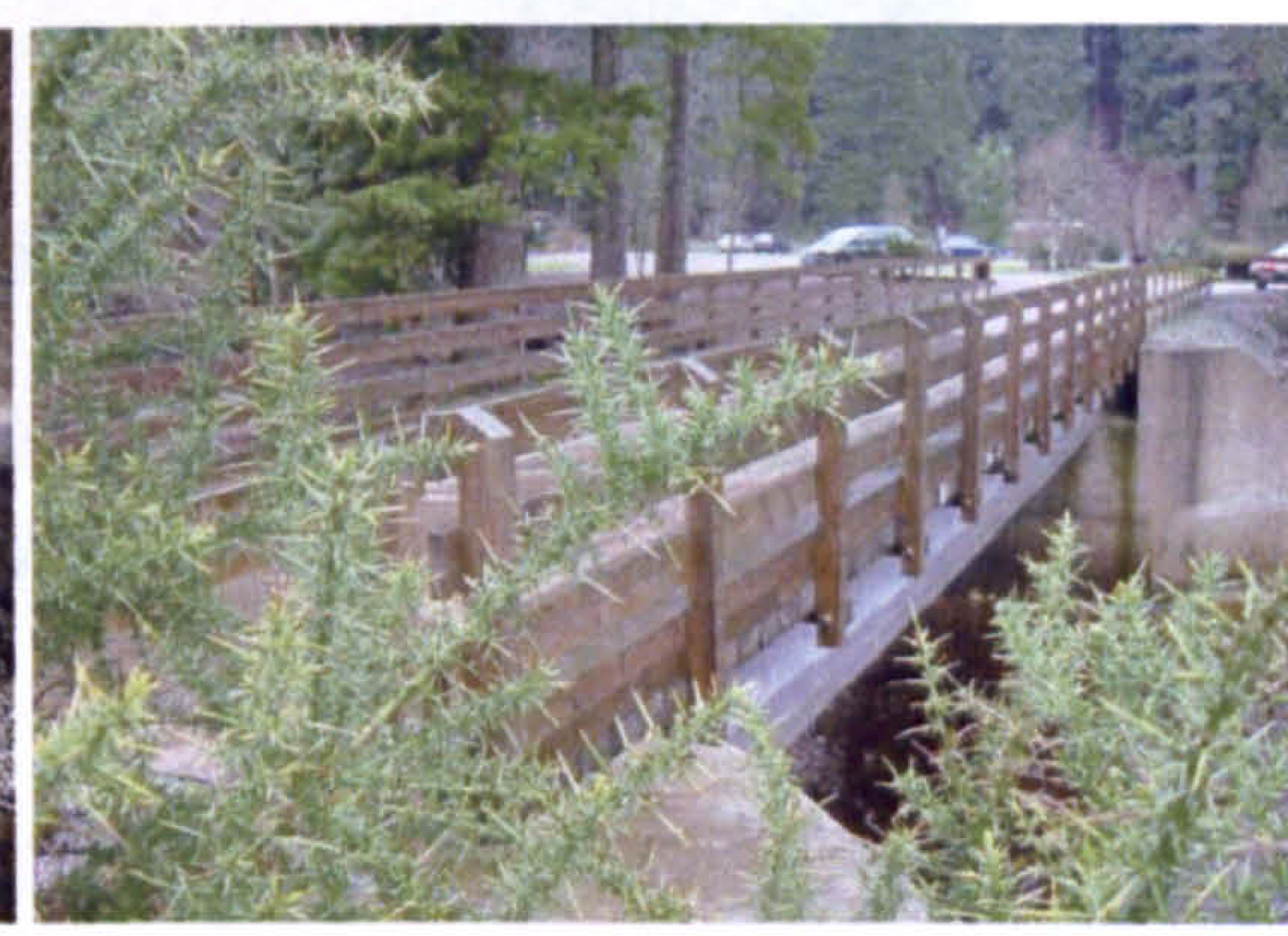



Fig 3.6 - Steel beam and timber deck bridges – Dalby and Benmore

3.4.3 Stress Laminated Timber

Because timber is the FC's commercial product, and in the interest of economy and good countryside aesthetics, there has always been a desire to construct timber vehicle bridges on the FC estate. This, however, was not possible with the small size of harvested timber in the UK. Work on vertical stress lamination became known in the UK about five years ago and a 4m span bridge was built as a prototype in 2001. A number of commercial bridges have now been built but the spans are restricted because the maximum timber size easily available in the UK is 250mm. The designs have been based on the Australian model, current UK codes of practice and Eurocode 5. Figure 3.7 shows a flat slab stress laminated bridge for large vehicles at the maximum span permitted by the timber sizes available. Before this design can be really useful, a method of creating a longer span is required. The current research is aimed at the design of arches supporting flat decks. In the USA and Australia they have built cellular decks for longer spans but these would not be suitable in the damp UK climate. This subject is discussed at length later in this report.

Fig 3.7 - Stress laminated timber vehicle bridge – Thames Chase

3.5 Footbridges

3.5.1 Deck Specification - Loading

BS 5400 [23] gives one footbridge loading of $5kN/m^2$ which produces strong, durable, vandal-proof structures but they are expensive and not elegant enough for the countryside. The Department for Transport has adopted this loading for bridges over roads. The FC is not legally bound by BS 5400 and uses loads from a joint publication with the Countryside Commission [19] which gives a countryside crowd loading at 2/3 of BS 5400 and a normal loading at ½. These loadings must accord with use and often footbridges have to carry livestock, cyclists or utility vehicles e.g. quad-bikes. Appendix 8 shows a table of loading for different uses of footbridges.

Design for dynamic loads is becoming more important as decks become more slender. The most public failure in this context was the Millennium Bridge, over the river Thames. Pedestrians transmit vertical and horizontal surges of load which can become a problem on slender and long span bridges. Design normally uses the static loads given in codes which are adequate unless the structure's fundamental natural frequency (FNF) lies within the range of the frequency of the applied loads. Walkers transmit load at about 1-2Hz while runners and jumpers transmit at about 2.5Hz, so to be safe, a footbridge should have an FNF above 5Hz which is what BS 5400 Pt2 Appendix C1 p46 requires. If the FNF is below 5Hz the acceleration must be within certain limits. Today, countryside structures are being designed within the danger zones so the effects of dynamic loads and the problems of resonance must be evaluated and considered in the design.

3.5.2 Parapets

Bridge use not only demands different specific vertical loadings, it defines the parapet design and the width of deck. Road bridges in forestry do not have containment barriers, which is one of the cost savings. This is justified because of the low speed limits on gravel roads in the forest. Bridge parapets are normally 1m high, unless there is a dangerous drop. The higher parapets needed for horses and cyclists are not necessary on a road bridge because these are 3.5m wide and the users do not need to go near the edge.

On non-vehicle bridges, horses impose high point loads from hooves, which affect deck boards [Appendix 8]. These animals need to feel secure, so horse bridges must have a high natural frequency. They, and other livestock, impose much greater lateral loads on parapets so posts must be designed to take these loads. On an English public right of way, parapets have to be 1.8m high for horses but 1.6m is accepted by the British Horse Society [26].

Generally, normal handrails are 1m high and have to sustain 0.74kN/m of horizontal line load. However parapets for crowd load, or where there is a drop of more than 2m, will be 1.2m high and be designed for 1.4kN/m of line load at handrail height. The parapet must be 1.4m high for cyclists. A further requirement in a relatively busy location is that the design should prevent people from climbing on the rails. This requires vertical infill panels or steel mesh over the rails. This must be arranged so that children and small pets are safe. The spacing of infill must be small enough to ensure a child does not get its head stuck but large enough so that it does not get a hand or finger stuck.

All of these requirements can interfere with the view for children and wheelchair users so the special design, shown in Figure 3.8, has been devised for an all- ability access bridges which caters for the combination of needs. Appendix 9 contains a useful spreadsheet which designs posts and rails for all combinations of use and loading.

Fig 3.8 - Specialised handrail design - Achray bridge

3.5.3 Choosing a Bridge Type

The choice of bridge will depend on span, use, cost and importantly, access. Unlike road bridges, footbridges are often built where there is no vehicle access and all materials have to be carried to site, which defines design. The availability of materials will affect cost and span and the ground conditions will affect abutment design. These many variables mean that a designer needs a range of designs to choose from and may make a decision e.g. to increase the span in order to lower the cost of abutments and thus affect the cost of the whole bridge.

3.5.4 Aesthetics

Aesthetics are a very important aspect of footbridges. These bridges are usually situated on walks which are intended as a countryside experience and, as such, become focal points for walkers to stop and look. A bridge in the countryside is always more that just a place to cross a river or a stop for enjoying the view. It often has a certain spiritual value – a place to contemplate – an aspect which can be very important to users.

"Like holding hands, a bridge embodies that feeling of forging a link across a void, bringing together what was once separate" [27].

With this in mind, every effort should be made to ensure that the appropriate bridge is chosen and that each particular location gets the bridge it deserves. Figure 3.9 shows a very good example of a bridge in context with its environment. It sits below the horizon with a shape to reflect the location. The material, timber, and its colour, merge with the surroundings to create a balanced aesthetic.

Fig 3.9 - Achray bridge Queen Elizabeth Forest Park – 17m span – completed 2005

3.6 Footbridge Types

The FC currently specifies three types of footbridge to cover most requirements. Occasionally a very long span is requested and suspension or cable-stay design is used, but that is the exception.

- Glentrool
- Aerial Mast
- Stress Laminated Timber

3.6.1 Glentrool

For many years the FC employed a standard design called the 'Galloway'. Its uniqueness involved the manner in which the deck and handrails fixed to the beams. As previously noted, timber beams which are split by fixings allow water to enter, causing rot, so a frame approach was developed in order to allow the deck support and handrails to be clamped onto the beams, thus avoiding puncture of the timber beams. The design reached the public domain and many hundreds were built by the army as well as estate owners etc. However, builders ended up taking short cuts and consequently, produced dangerous bridges. A very important lesson was learned about those designs - which ones must be controlled by Engineers and which ones can be left to amateurs. After this became apparent, the 'Glentrool' was developed as a replacement for the Galloway, Figures 3.10a and 3.10b.

The Glentrool utilizes steel or timber beams with steel angles above and below the beams which project out to support the handrail. This is a torsional restrain mechanism designed and patented by the Author, as part of the Aerial Mast Bridge project (Section 3.6.2). The deck and handrails are always timber and the beams are usually steel. The main design philosophies are to allow air flow around timber joints to avoid rot and for a design which is easy to build on site and cannot result in a dangerous structure, even if short cuts are taken. This design is very economic and is usually first choice for spans up to 8m span. For longer spans, the beams become difficult to handle and unless there is vehicle access to site, another design may be preferred.

The design does not include any diagonal bracing and it has been suggested that it could fail through lateral torsional buckling, as the lateral angles do not provide sufficient restraint. However, the structure is fixed at the supports and acts as a grillage so it does not, in fact, need bracing which would be, in any case, difficult and costly to fit.

Fig 3.10a - Glentrool bridge - Symmond Yat

Fig 3.10b - Glentrool bridge – Pucks Glen

3.6.2 Aerial Mast

This design uses factory- produced triangular steel truss units, 3m long which are bolted together on site to produce beams of length up to 24m. The base and side of the beam is 500mm wide for spans up to 15m and for spans between 15m and 24m the side is increased to 750mm. Beam units are laid side by side to produce a bridge of the desired width and they are linked laterally using steel angles 'U' bolted to the top and bottom tubes of the mast sections. A timber deck is fixed to the top of the beams, and handrails are attached to the lateral steel angles as shown in Figure 3.11.

Fig 3.11 - Milton bridge – near Aberfoyle

The advantages of this design are:

- Every piece of the structure can be carried to site.
- It is very lightweight.
- It is easy to build.
- It has good torsional resistance.

This design is very popular with Local Authorities and countryside groups because of the ease of construction. All the component parts can be carried by two men to, often, inaccessible locations. More than one hundred of these bridges have been built in the last ten years since the design was patented by the FC, with the Author named as Inventor.

Figure 3.12 illustrates the diversity of the system. One bridge of 20m span was erected in one week, to replace a stone arch washed away in a flood just before a major cycling event. The other is on an SSSI on a beach and the steel was double dip galvanized for extra protection. After seven years of use, this bridge has weathered, so that it can barely be seen from a distance and is in perfect condition.

Fig 3.12 - Aerial mast bridges – 25m span in Wales and 15m span in Elie, Scotland

3.6.3 Stress Laminated Timber

This is a relatively new design and the technical advantages are dealt with in detail in later chapters. The design was developed first as part of an overall plan to use more timber in rural construction for three reasons:

- to utilize the FC product
- to increase the Public's confidence in timber as a structural material so they might consider a timber frame house.
- to improve the aesthetics of the countryside.

These bridges can be built as simply supported flat deck spans, as shown in Figure 3.13 or as two-pin flat arches which are not too steep to walk over, as shown in Figure 3.14. The flat slabs use more timber for the same spans but do not need such large abutments. The arches are very slender and have good visual appeal, but require sound foundations.

Fig 3.13 - Pucks Glen flat SLT bridge - Argyll

About thirty permanent bridges have been built as part of the development programme of this project. They are becoming very popular and the demand has surpassed contractor capability so new contractors are being tutored in the construction techniques. Constructing bridges in parallel with laboratory research provides many opportunities for performance and proof testing. Already, the cost of building the bridges is reducing as confidence and knowledge increases. Although the aim is to build long span timber road bridges, there will always be a demand for this design of timber footbridge.

The main appeal, apart from the low cost of materials, is the rigidity in the arch structures which have a span to depth ratio of up to1 to 100. The bridges are developing a great deal of Public interest and achieving their goal of becoming focal points. The stress lamination technique will lead to many derivatives of similar design in the countryside. Research is on-going to try to find simple alternatives to the costly Dywidag [28] stressing bars and the hydraulic jacking process. Threaded bars have been used for small bridges in the laboratory where 2 tonnes of tension was sufficient. The long span bridges have proved to be very popular but it is the span range around 10m which will prove to be the most economic. Small bridges of that size are very lightweight and can be constructed off-site or on the river bank, and then lifted into place.

Fig 3.14 - Forth bridge – 20m Span – Flanders Moss

Fig 3.15 - 9m Span bridge – Newcastleton

3.7 Culverts

Culverts are commonly used under forestry roads for small catchments and relatively low flows. They are always used for taking roadside drain water but their use is not regarded as good practice in natural watercourses because of potentially detrimental effects on fish.

3.7.1 Circular Pipes

The smaller diameters were all made from concrete until ten years ago, when ribbed plastic pipes were introduced, Figure 3.17. Now these twin walled culverts are the first choice because they are light to handle and quick to install. They are not as strong as the concrete pipes and need to have the backfill properly compacted at the sides of the pipe before the top hardcore cover is placed, otherwise the pipe will be squashed. Some galvanized corrugated steel pipes, Figure 3.16, are still used for larger diameters but in acidic soils they can corrode quite quickly. Circular pipes are not recommended for natural watercourses unless they are sunk well below the stream bed, thus avoiding interference with the natural gradient and gravel makeup of the river bed.

Fig 3.16 - Corrugated steel culvert – corroding and too high above the stream bed

Fig 3.17 - Twin walled plastic culvert

3.7.2 Box Culverts

These are factory-produced, rectangular boxes laid side by side to form a span over a river. Traffic usually runs on a reinforced concrete slab, cast over the top of the boxes. They can also have an adverse affect on the passage of fish up and down a river so, again, they must be sunk into the bed. Some designs have protrusions on the river bed face to catch migrating gravel and form a natural river bed for fish, Figure 3.18. These structures can be useful in special circumstances but a bridge is always the best solution if a good site can be found. Bridges leave the bed untouched, resulting in zero impact on the wildlife environment.

Fig 3.18 - Fish friendly box culvert

3.7.3 Pipe Arches

Fig 3.19 - Corrugated steel pipe arch Sweden

This is an innovative idea recently researched by 'FERIC' (Forestry Engineering Research Institution of Canada) [29] in Canada and some examples were observed by the Author, in Sweden, Figure 3.19. It is a useful crossing for forestry or minor public roads, where the height from the road to the river bed is not great enough to allow a full pipe.

Proprietary pipe arches by "Armco" have been available for many years but are expensive and are therefore only economic for very large spans. On the other hand, circular pipes have been used in diameters up to 2400mm, often causing a barrier to fish and severe downstream erosion. A circular pipe can be installed with its invert below the stream bed, but this usually involves cutting some rock, is difficult to do and can cause great disturbance.

This most recent development involves slicing a large diameter circular pipe along its length and using the cut section as an arch. Only half the length of circular pipe is needed as both pieces are used with a small overlap at the centre. This was considered many years ago but the problem of bearing at the springings has always been perceived as insurmountable, at the correct cost. The proprietary Armco arch specifies a concrete kerb which is difficult to cast without the risk of pollution. The new proposal is to lay the cut edge into a galvanised steel channel section. This is not so much a mathematically designed solution, but a practical one which has been found to work.

In the UK some trials are necessary to establish if a plastic pipe arch will sit in a channel without fixing. In Sweden, steel arches were used and they were tack-welded to the channels.

3.8 Abutments and Piers

Abutments are built to support the bridge and hold back the soil of the approaching road or path. For that reason, abutments often have wing walls attached to them. Their other main function is to raise the bridge high enough to avoid flood damage and to protect the foundation of the bridge. A support between abutments is a called a pier. This is positioned to shorten the beam span and usually has only to support a vertical load without being undermined. Bank seats are simple abutments built at the top of a slope which usually supersedes the requirement for wing walls.

Abutments are built using four types of materials/construction -

- concrete
- gabion
- reinforced earth - geotextile
- timber

- each one has its specific advantages and disadvantages.

The choice of abutment for a specific situation will depend on:

- volume of traffic
- bearing capacity of the ground
- type of bridge it has to support
- availability of ready mixed concrete
- availability of skilled labour
- height of abutment
- aesthetics.

The design bearing capacity of the ground is required, whichever abutment type is chosen. In Forestry and Agriculture, the cost of site investigation is usually not justified for small rural bridges so the likely bearing capacity is assessed by an experienced Engineer and the design is based on that. If the bridge is on high ground, the subsoil is most likely to be rock or a glacial mix of boulders and clay and it is usually safe to take a bearing capacity of about 300kN/m^2. In lower ground, if the bridge is in a very flat area which is obviously an old flood plain, the maximum recommended is about 150kN/m^2. Settlement must also be considered in poor ground. Piers should only be built on rock outcrops for small bridges. The Engineer needs to be present when excavation takes place to ensure the assumed bearing capacity is achieved. New environmental regulations in 2006 by the Scottish Environmental Protection Agency [30] will make it very difficult to obtain permission to build a pier for a small bridge.

Bridge abutments are important structures from an environmental perspective. They provide sheltered areas for nesting. Certain species of bird particularly enjoy the secluded bearing areas, so it is good practice to build in a nesting box, Figure 3.20. It should be cast into the concrete with a slope outwards for drainage and have a small retaining strap to hold the nesting material. It is also good practice to leave little slits about 20mm wide and 200mm long for bats to sleep in.

Fig 3.20 - Picture of nesting box

3.8.1 Concrete

This is the most durable, but unfortunately the most expensive material and on sustainability grounds, by far the most energy intensive. The manufacture of 1 tonne of cement creates 1 tonne of CO_2. A useful rule of thumb is that, for a 10m span, the low-cost bridge deck and its abutments are roughly equal in capital cost. As the span decreases, the proportional cost of the abutment increases. Mass concrete is always used because the cost of steel fixing to produce reinforced concrete abutments negates the savings in material, and skilled labour is not often readily available in rural areas. They lend themselves well to the addition of wing walls which are always necessary to support the road next to a bridge. Bare concrete looks unnatural in the countryside and in sensitive areas, should be hidden by stone facings, Figure 3.21. Although these are expensive, if they are used as permanent shutters the cost can be offset. This is only possible if small concrete lifts can be justified as the wall would have to be extremely strong to resist the pressure of a large pour. If ready mixed concrete is available, it is difficult to recommend anything other than a mass concrete abutment, especially for a permanent heavily used bridge. Figure 3.22 shows two typical examples of these abutments and their lack of visual appeal.

Fig 3.21 - Stone faced concrete abutment – Carribber bridge

Fig 3.22 - Concrete abutments

3.8.2 Gabion

These are wire baskets filled with selected size and quality rock. The durability of the baskets is always in question, when used as permanent abutments. In non-acidic water and without undue abrasion or indeed as bank seats, they should last the fifty-year life of the bridge.

Their great attractions are:

- cost – half that of a concrete abutment
- aesthetics – look more pleasing than concrete – a matter of opinion
- safety – no risk of polluting the water with concrete
- fill material is often available on site.

Their disadvantages are -

- life of the protective coating
- slumping and bulging as a result of poor quality filling
- span of bridge must be increased to keep bearing away from front face
- risk of vandalism
- cost of hand placement
- soft rock fill can dissolve or erode

In practice, gabions often cost as much as mass concrete because of the hand placing of stone in the faces. If limestone or other chemically or dynamically unstable fill is used, it will wash away and the basket will slump, especially if the BRC [31] stiff box type is used as opposed to the Maccaferri [32] basket, which is designed to slump and nestle. Vehicle bridges must have a substantial sill beam for the beams to bear on so that load spread is assured. Differential settlement is always possible with gabion abutments. This sill beam must be about 500mm back from the front face in case the wire on the vulnerable front face is cut or bursts. Catastrophic collapse could result from bad detailing of bearings. Vandalism is a problem near urban areas so this form of construction should be avoided near towns. Figure 3.23 shows a good example of well filled baskets but the sill beam is closer to the front face than would normally be recommended. This places extra stress on the front face.

Fig 3.23 - BRC Gabion abutments – sill beam too far forward

3.8.3 Reinforced Earth

This is a relatively new form of abutment for rural bridges but one with great potential for the FC or minor public road bridges. These are usually formed from pre-cast concrete face units about 150mm thick held back by steel bars, webbing or a matrix of geo-grid. As the face is built up, each unit is tied back and selected fill is compacted over the ties. When the abutment reaches full height a concrete slab is cast on top as the bearing for the beams.

This is an excellent form of construction where ready mixed concrete is not available and good gravel fill is readily available - a common combination in upland Scotland on FC land. It is also the best solution for high abutments which are now becoming necessary with the construction of special bridges for wind farm access. They have to be built to suit shallow, steady gradients for the large cranes and the transport of generators and propellers. Normally, economy demands that abutment heights are controlled by the necessary flood waterway area and road gradients are designed to fit.

Although pre-cast concrete face units are most common on major highway bridges gabions are more cost effective in rural situations. This means that geotextiles are best placed at 500mm between layers to coincide with the gabion layers. Trials will begin soon using used tyres as the front face, which could produce the most sustainable abutment ever used on FC land. However, the aesthetics will be poor and the correct locations will have to be chosen carefully.

Figure 3.24 shows the geotextile in a roll and it can also be seen in place just below the large boulder. Figure 3.25 shows a large abutment built for a 700 tonne crane to erect wind turbines. After carrying several hundred thousand tonnes of material, over a very short space of time, these bridges suffered no damage.

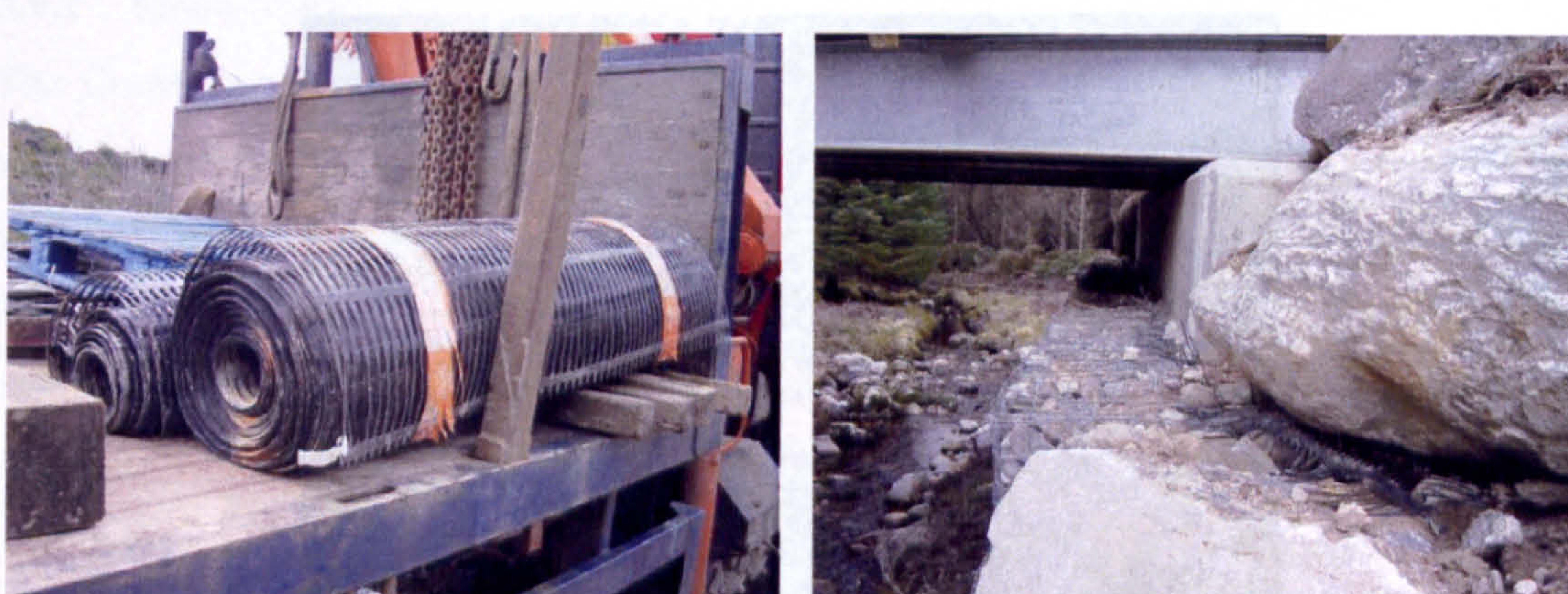


Fig 3.24 - Geotextile for abutments at Farr

Fig 3.25 - Gabion abutment face to a geotextile abutment – Farr Wind Farm

3.8.4 Timber

Timber abutments were used extensively in the New Forest in the South of England, where durable hardwoods were available. Many now require to be replaced but only softwood is available. Although it has to be pressure treated, it is available at low cost. Past systems have basically comprised a front face of abutment and wing walls, filled behind with hardcore, Figure 3.26. However, modern designs will incorporate some form of tie back using a geogrid or timber ties like a Criblock [33] system. Most of these systems could suffer wash-out of the retained material if they are used in a fast flowing river, so location will have to be chosen carefully.

Fig 3.26 - New Forest bridge abutment

A trial system was exhibited by Forestry Civil Engineering (FCE) [12] at an agricultural show in 2001 and tested under a new initiative called Innovative Timber Engineering in the Countryside (InTEC) [9], Figure 4.3. Construction is from sawn pine with chemical pressure treatment, after all drilling and cutting is complete. These abutments are most likely to be used where aesthetic considerations are high on the agenda. There will always be a risk of premature failure through rot, even with chemical treatment.

3.9 Other Factors Affecting Bridge Design

It is not enough to measure the width of a river and build a bridge strong enough to take the loads which need to cross. Today there are many other factors to be considered. Some are simply good practice but others involve legislation.
Included in this category are:

- flood design
- environmental aspects
- fish
- energy equations and renewable materials
- recycled materials
- sustainability and whole life costs.

3.9.1 Flood Design

The Centre for Ecology and Hydrology - Flood Estimation Handbook [34] is usually sufficient to calculate the maximum flood which is used to calculate the bridge height and span in the UK. In 1999 this publication replaced its predecessor, the Flood Studies Report – 1975, but recent climate change, perceptions have demanded some revisions which are expected early in 2006. Even with revisions in the guidance, the FC has decided to calculate bridges on a 1 in 100 year return period and culverts on a 1 in 50 year return period. However, these calculation methods are statistically based and specific river information is more valuable. Careful research is needed to trace past floods. Some of this can be done by local observations. Evidence of past flooding can often be observed in the form of residue marks on e.g. old church walls. Much flood information can also be gleaned by questioning local elderly residents, who can often provide much valuable data. This information, together with statistical evidence and cost calculations regarding the consequences of a flood, is required in order to make an informed decision.

3.9.2 Environmental Design

Rural structures need to harmonise with their surroundings in colour, scale and texture. This is usually achieved, where possible, by using local materials for the structure or in order to hide unnatural elements.

Local groundwater supports the natural flora and fauna of an area so, if it is drastically changed by a bridge or its approach road, it can upset the balance of an entire area. Wetlands should be avoided, as they are the most valuable and vulnerable. If they must be crossed, water must not be channelled. Instead, a boardwalk or a multi-pipe crossing should be built, so that the water can continue on its historic path and nurture the wildlife, Figure 3.27.

Fig 3.27 - New Forest pipe bridge

There may be archaeological remains in the area which should either, not be disturbed or be catalogued before they are disturbed, or even spanned over with a bridge, Figure 3.28.

Fig 3.28 - SLT over woodbank

Local habitats must always be considered and Local Authority environment officers consulted before embarking on any bridge project. Badgers e.g. may live in the area and, as they need a run, bridge abutments require to be built back from the river to allow them clear passage at the water's edge.

Care of the fish life is important and fish are protected by legislation from EA [35] and SEPA [30]. The basic rule is to avoid interference with the river and its bed during construction or as a consequence of the design. SEPA publishes excellent guidance, so there is no need for ignorance. In Spring 2006 a license will be necessary prior to building a bridge to ensure that damage can be minimized. This will not stifle activity because an excavator may, indeed, work standing in the river but it must be free from oil leaks and outwith the fish breeding season. The most common forms of environmental vandalism, in fish terms, are the creation of waterfalls with round culverts, or paving the bed under the bridge at a level higher than the natural bed. It is better to use arch culverts, not to pave and concrete box culverts can have nodules cast onto the base, or rough masonry, to catch migrating gravel which acts as a natural bed for fish, Figure 3.18.

3.9.3 Sustainable Bridge Design

This is fast becoming the most important element of design because of the political implications. The funding for a project often comes from a well lobbied source which has made manifesto promises to an electorate. Recreation projects are all about health and education so bad examples must not be observable within the infrastructure. Commercial infrastructure will always offend someone, so if the environment is not offended the opportunities for objection are diminished.

Factors that must be considered are: –

- energy efficiency
- material selection
- construction waste reduction
- air quality
- the visual environment and
- the general health and well-being of the users.

This list is not exhaustive but these are the most important factors.

A low energy structure should employ natural materials which have not had extensive processing and manufacturing inputs. Concrete (cement) and steel are high energy components, while masonry and timber are low ones. A well designed, sustainable, rural structure will use the minimum quantity of energy. When making these decisions it is important to remember hidden costs and not try to build the justification which is desired. These are common faults in 'sustainable' decision making.

It is important to remember to include transport of materials - a major issue with timber which starts off with a zero energy rating. However, due to its bulk, transport fuel energy becomes a very large portion of the equation. On the other hand >50% of steel production in the UK is from recycled metal and cement production is now burning used tyres where they can.

Unfortunately, in our 'short- term Society' it is usually the minimum capital cost which matters most in the choice of structure, whereas the lowest whole life cost should be the criterion. A Life Cycle Analysis would take into account factors such as maintenance, manufacturing energy and eventual demolition. This sort of analysis very often shows that the lowest initial cost is not the lowest total cost. However, business is usually judged on this year's balance sheet, so it is up to the Engineer to achieve the best balance between sustainability and cost.

Key factors are:

- using timber for main structures,
- keeping concrete use to a minimum,
- using steel where it is only really needed,
- recycling materials where possible.

Fig. 3.29 shows a typical simple log bridge.

Fig 3.29 – Sustainable log bridge

3.10 Costs of Forestry Bridges

Exact costs for forestry or minor public road bridges are almost impossible to estimate because of particular site conditions, availability of contractors and fluctuating materials costs. However average costs with relative comparisons are useful to help make choices.

Factors affecting costs in the rural areas fluctuate with time but today the main ones are:

- the wind farm projects competing for the local contractors
- world steel prices
- fuel and energy costs affecting delivery
- concrete prices.

A steel beam and timber deck vehicle bridge should cost the same as a pre-stressed concrete one - if beams are available - and should be about £800 per square metre of deck. This should increase for short spans because two abutments are still required and they will be roughly the same size, whatever the span. Similarly, the beams are heavier for long spans which will increase cost. The optimum span for cost is about 10m span where abutments and deck costs are approximately equal.

A stress laminated timber deck should cost about half of the other two alternatives at construction, but the whole life cost may be the same, because its life may be about half. On the other hand plans could change in twenty five years and perhaps the bridge may not be needed for fifty or one hundred years, so the true cost in that case would be half. Again, on the other hand, if waterproofing of the deck is well maintained, pressure SLT decks should last fifty years. Abutments will cost about the same as for the other designs of deck.

Concrete abutments are about half the cost of an average 10m span bridge; gabion abutments are about half the cost of concrete ones. Timber abutments could be a little less expensive than using gabions and geotextile abutments, using old tyres, may become the best value in the future.

Footbridge decks cost less than heavy vehicle decks, which makes the overall cost more sensitive to the abutment cost. The secret of economic footbridge design is to keep the abutments simple, which means keeping them away from water and building them on the bank if possible. It is usually more economical to extend a footbridge span rather than build an expensive abutment. If these rules are followed, footbridges should cost about £600 per square meter of deck.

The above estimates are capital costs and perhaps it would be more sensible to consider Life Cycle Analysis. This approach shows true costs and highlights the value of using timber or recycled materials. Many attempts have been made at this approach and most are skewed to show answers which will sell products. BRE has carried out research to design a system which takes account of everything. It even puts a value on the amenity loss of cutting a tree down to produce timber. These obscure values were derived from asking the public what they thought, so they are not just professional judgements. Although a Life Cycle Analysis pricing exercise is very involved, it is something that political bodies like Local Authorities will have to do very soon to justify their expenditure.

A third approach to the cost of a bridge is the energy value of the structure, which results in a sustainability value. This can be calculated as an amount of CO_2 exhausted to the atmosphere during the manufacture of the bridge components and the site construction. This approach would show the real value of a timber bridge on recycled tyre and geotextile abutments over e.g. an aluminium bridge on concrete abutments.

The perfect cost exercise would show the results of all three approaches and the compromise which suits all aspects of the design brief would be chosen.

3.11 Conclusions on the Use of Forestry Bridge Designs on Minor Public Roads

There is an established departure from the strict requirements BS 5400 and DfT (Department for Transport) memoranda in the design of forestry bridges without compromising safety. Introducing more timber into the designs, especially as primary structural members, will be an extension of that established practice. Further, as a result of an initial approach, the DfT is awaiting submission of SLT designs with a view to permitting them on public roads. In the meantime the Brechoille model Figure 3.3, where a bypass is built on private land alongside the public road bridge, is a suitable compromise.

Many of the fragile masonry arches on minor public roads need to be replaced, especially if they are to take forestry traffic. Quite a number of arches are strong enough to take the occasional large lorry but, as previously mentioned, they need time to 'settle' before being heavily loaded again. This is not possible during harvesting. The tried-and-tested forestry solutions offer a credible low cost alternative to the cost-crippling specifications of Central Government for public roads.

The deviations from the current Standards should be evaluated in terms of risk and the speed limits. It is a fact that drivers in Sweden slow down on unsurfaced roads, resulting in fewer accidents. The same may become true in the UK and, if so, the forestry- type bridges without crash barrier parapets could become appropriate.

The overriding factor for the use of lower specification structures in the forest is cost. Traditionally, this has been calculated as capital cost but now the sustainability and energy issues are important, especially in an industry which preaches clean activities. Recreation, forestry, sustainability, environment, and emissions all go hand in hand which means that the use of steel and concrete need to be kept to a minimum. The use of timber must be maximized by using it for primary structural members. This will require large sections from small trees so lamination of some kind is required. Gluing and truss formation have their difficulties which leaves mechanical stress lamination. This form of construction has not been used in the UK so there is a need to assimilate as much past foreign research work as possible and carry out developments which are necessary to translate this form of timber engineering to UK codes and site conditions. Cellular boxes are not suitable for the UK but arches supporting flat decks are.

Rural profitability is a finely balanced equation and an important aspect of this entire initiative is to find an appropriate and sensible solution to enhance access to the countryside and protect employment. This could mean converting potholed, tarred roads into gravel roads and introducing stress laminated timber arch bridges to rural public roads. Environmental impact would be reduced, alongside cost and emissions.

CHAPTER 4

4 REVIEW OF STRESS LAMINATED TIMBER BRIDGE DECKS

4.1 Historical Development of Stress Lamination

Timber has a good strength to weight ratio and it is easily worked and easy to fix on to. However, being a natural laminate, it is orthotropic, contains natural defects and is really only useful as straight lengths. Its best properties are its tensile and compressive (end bearing) strength parallel to the direction of grain. Although it is fairly good in bending it is, however, poor in bearing at right angles to the grain. These properties made it the natural choice for pit props in mining but today, that market is greatly diminished. In the days before steel and concrete it was the only spanning material, so it was used for roofs and floors, in bending. It is still mostly used in bending today but the poorer quality, quick growing, material from the UK does not excel in that market. Nevertheless, the poorer quality material is still excellent in compression, so an arch structure would be a natural choice. However, trees usually tend to grow straight.

To make an arch or dome from straight, or slightly curved, pieces of timber, many short lengths are needed. This means many joints have to be made and, because of timber's poor bearing capacity across the grain, highly stressed joints are not easy to make. It was probably that thought process which inspired Philibert Delorme [36], the father of French Neo-Classical architecture, to build the first known timber dome, using mechanical stress laminated timber, in 1561. He described the technique in his treatise, 'Nouvelle Inventions Pour Bien Bastir et a Petits Frais'. It comprised a series of short curved timbers laid side by side with staggered laps. This would ensure a significant cost saving over a masonry dome and it would be so lightweight it would not need the same amount of support. He used a system of clamps and wedges to compress the laminates and analysis would have shown no areas of significant stress concentration. Figure 4.1 shows an example of an early arch.

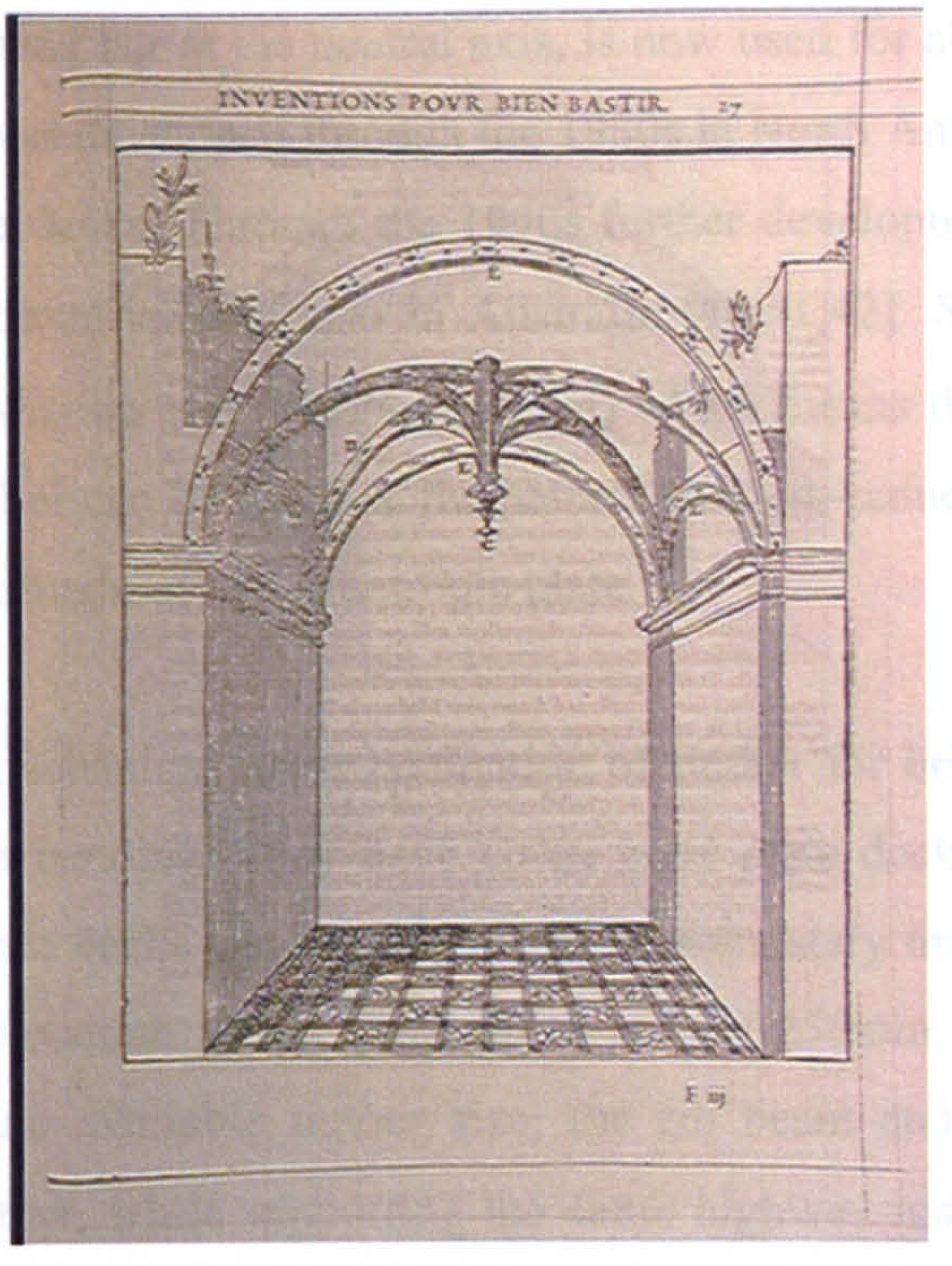


Fig 4.1 – Philbert De Lorme arch [36]

The idea was used in early railway bridges where a complete deck slab was formed by nailing the laminations through to two neighbouring laminates. It was especially popular in Canada and the USA in the late 1800s but with time, these decks suffered badly when nails corroded and became loose. The concept of prestressing (post tensioning) laminated timber bridge decks was conceived in Ontario, Canada in 1976, as a method of upgrading these existing, deteriorated, nail laminated wood decks. The first bridge deck to be repaired was at Hebert Creek in Ontario [37]. The deck was 'squeezed' back together, using transverse post-tensioning. This was done by adding a steel channel to each edge and connecting them with transverse steel bars. The bars were stressed, thus replacing the need for the nails connecting the laminates. This developed into a new construction technique whereby the transverse bars were passed through holes in the timbers at their neutral axis, thus avoiding external bars which were being damaged by lorry wheels running over them.

This technique, a stressing bar at the neutral axis, is now used for all modern mechanical stress lamination and was developed through the 1980s in North America [38,39], mainly for replacement bridge decks. Through the 1990s further developments were made in a number of European countries and also in Australia [40,41,42]. Stress lamination is a very efficient way of sharing and distributing load, which means that low grade lengths of variable quality timber can be used, as the natural strength-reducing characteristics of timber are dispersed throughout the orthotropic plate.

All developments in mechanical stress-lamination of timber for bridge decks have used flat decks or beams in bending. They have either been plate decks, rib beam decks or cellular decks. The plate decks can only span to approximately 6m, using full highway loading and normal maximum timber sizes up to around 250mm deep. Because of the restriction on maximum available timber size, the rib beam and cellular decks were developed to span further, while supporting the same highway loads [40,42]. These rib decks require laminated veneer lumber beams and the cellular decks entrap moist air which can create a rot problem. Consequently, neither is suitable for use in the UK.

Prior to mid 2002 there had been no known examples of stress-laminated timber bridge structures in the UK. Initial investigation was prompted by a need for low cost forestry and rural public road bridges which had originally been built as stone arches and traditionally replaced by steel and concrete. Home-grown timber is now plentiful in the UK but the quality and sizes are limited, so mechanical stress lamination techniques, similar to those used in the USA and Australia, looked to be of interest.

Before work began in the UK, the span limitations of slab decks were acknowledged. The cellular decks [42], composite with inverted steel "T" beams [43] or LVL, and stressed glued-laminated beams [44] were all known to be of no interest in the UK, for various reasons. The rejection of these longer span derivatives was based on cost and climate. There is no 'Glulam' or LVL industry in the UK, so they would have to be imported. The UK is much wetter than other places where SLT decks have been used and UK Design demands that timber structures are designed in such a way that they are able to air-dry between wet periods.

This led the Author to investigate implementation of the stronger engineering properties of timber (compression and end bearing), in an arching action. This would, ideally, overcome the limitations of decks in bending and create structures through which air could flow.

4.2 Recent International Developments in Stress Lamination

The mechanical stress lamination technique was adopted in the USA at a time when a national survey showed a need for many small-span replacement bridges in rural North America [45]. The form of construction seemed ideal, as it utilised timber in smaller sections which were readily available. Further, the construction did not require very specialist skills. The Timber Bridge Initiative was passed by Congress in 1989 [46]. This emphasised the need for research into new bridging designs and, of course, brought with it, funding. The initiative was to utilise wood and provide rural highway infrastructure to replace or repair many of the 592,000 bridges which make up the Federal Highway System. Of these bridges, 40,000 are all timber and a further 40,000 have timber decks. The forests have a further 7,500 bridges, half of which are timber. A major programme of research and trials was set up by the United States Dept of Agriculture (USDA) Forest Products Laboratory, in 1988, to develop the mechanical stress lamination decks [Ritter, M.A., 1990] [38]. This eventually led to a new American Association of State Highways & Transportation Officials (AASHTO) standard on the subject, in 1991. From those beginnings the USDA's National Wood in Transportation Program has funded 322 projects, resulting in many more timber bridges throughout the USA.

As development took off in the USA, (four hundred bridges in the first seven years), Michael Ritter visited other research centres in Australia and Europe. He stimulated development in a number of Institutions, including the Sydney Institute of Technology and the Nordic Timber Council. In 1996 Crews, K.I. published a guide for Australian practice and Kleppe, O. and Aashiem, E. produced some spectacular structures in

Norway. Other parts of Europe have also benefited from this form of construction

The Norwegian government has invested heavily in this system and in more complex derivatives of the basic slab. Typically, they have formed Laminated Veneer Lumber beams and stress laminated them to form a slab. They have built a number of impressive timber bridges on the public road network, even on motorways, Figure 4.2. Indeed, five to ten per cent of all bridges built in 1998-99 were of timber.

Fig 4.2 - Timber "A" frame and SLT deck bridge near Oslo

The bridges are well detailed and consideration has been given to all aspects of durability and deterioration. They are pressure- treated with Creosote, providing timber with the required treatment for a life of one hundred years. The preservative treatment is, however, of greater than normal significance. Dimensional stability of these decks is critical in order that the pre-stress remains during service when the timber is wet and dry.

The correct treatment can help reduce movement to ±0.25%. In Norway the timber is kiln-dried to 14% before treatment, which increases treatment volumes and reduces shrinkage and swelling in service. In the UK and many other European countries, the use of Creosote is strictly controlled because of its toxicity and liability to leakage in its first year of service. It is understood that special care will be needed, but the returns should justify its use.

This flat slab SLT design has a number of perceived advantages:

- it utilises the industry product
- timber is plentiful and inexpensive at present
- lightweight
- ease of fixing parapets
- low expansion and contraction compared to steel and concrete, allowing continuous bitumen surfacing without joints
- it can be factory produced.

The disadvantages are likely to be:

- restricted to shorter spans
- regular maintenance when timber shrinks, due to drying
- the stressing systems are not readily available in the UK.

The UK is starting late in this field of bridging and there is much to be learned from the US, Australia and Norway, but the first arches were developed here. Links with the international researchers will be invaluable over the next five years while this unique form of construction develops in the UK.

4.3 UK Developments in Stress Laminated Bridges

There are very few research bases dealing with timber engineering in the UK and only Napier University's Centre for Timber Engineering is examining Stress Lamination in bridges. Chapter 2, Section 2.3 of this report makes reference to this.

The Author designed a pseudo stress laminated footbridge for an innovation centre ten years ago, following a discussion with Coed Cymru in Wales, who were looking for markets for their short, small, round hardwood. This stimulated an interest in the subject and in Timber Engineering, in general. From that, emerged InTEC, the New Age Flitch Beam [47] and the first stress laminated timber arch bridges. The timber arch was developed as an idea, previously untried, to perhaps find a way of building long span vehicle bridges, suitable for the UK. It also provided the UK research effort with something to offer the international stress lamination fraternity, in return for their results on flat decks, which will be very useful in the UK.

Research has not been well funded, and even following the Author's DTI International Technology Mission Report [4], road authorities and Central Government have been reluctant to embrace the idea of timber vehicle bridges. The need has certainly been greater in other countries, but SLT bridges on UK roads could save money and reduce CO_2 emissions. Some home research results will help confidence and already there are signs that published papers are having an effect.

The Author has built about thirty permanent SLT bridges and is presently monitoring tensions and performance, with a view to extending confidence not only in SLT, but in the use of timber for other structures. These include 4No 20m span arch footbridges, an 8m span road bridge, 2No 7.5m span Forwarder bridges and a number smaller span footbridges. The aim is to make the work of this thesis the foundation for innovative long span road bridges and footbridges up to 25m span.

The first display bridge was built for the Royal Highland Show (RHS) in June 2000, Figure 4.3, and this was followed by a 6m span arch in 2002. This was tested at the newly formed Centre for Timber Engineering and spawned the beginnings of this research programme.

Fig 4.3 - Vehicle bridge with Macaloy bars

The next test bridge was a 15m span commercial bridge built in the laboratory, tested, and then transported to its permanent site 250 miles south. This inspired confidence, so a 20m span test bridge was designed and built at the RHS in June 2004. This was then moved to a forest location for rigorous testing, using static and dynamic loading.

This thesis details the findings of all of these tests and many others. The conclusions will pave the way for future innovation in stress laminated structures for bridges and buildings.

The precise details of the test bridges will be given in the proceeding chapters but generally, the loadings were the same as for steel and concrete bridges and a range of timber grades were used to demonstrate the suitable bearing capacity of the low grades. Hardwoods have been used for outer laminates, but all inner laminates have been softwood.

CHAPTER 5

5 CHARACTERISTICS OF SLT BRIDGE DECKS

This section explains the details of what is already known about stress lamination, in other countries around the world. It is intended to provide a full technical background to the subject, within the scope of this thesis. Previous chapters have provided information about the history of development but little about the details of the structural actions of this new form of Timber Engineering. Each aspect of stress lamination will be explained and interpreted in relation to arches.

5.1 Fundamentals of Stress Laminated Timber Decks

The decks are vertically laminated and stressed together without the use of glue, thus avoiding many quality control issues concerning moisture content and smooth surfaces. It permits cost effective bridge decks in the UK, where an established glue laminated industry does not exist. In the USA it is common practice to plane one side in order to ensure even laminate thickness but the Author has found that a good sawmill will produce sawn laminates of near enough equal thickness and the sawn surfaces have a higher coefficient of friction.

The laminates run with the span and the stressing bars are transverse. Together, they create the structure and the deck. The tension bars provide enough force to create friction between the laminates which transfers the vertical shear to the supports. The initial compressive stress between the laminates is approximately $1N/mm^2$. This has been used in USA and Australia and Eurocode 5 Section 6.1.2 (6) states that $0.35N/mm^2$ residual prestress can be assumed for an initial prestress of $1N/mm^2$. The lateral tension also creates an orthotropic lateral load sharing plate to distribute point loads to the full width of the structure. This effect is not as critical in an arch footbridge, with pedestrian load, as it is on a flat vehicle bridge with wheel loads which cause prying of the laminates and high local shears [48].

The timbers can be full length for short flat spans and, where possible, bridges should be built this way. This is not possible for longer spans or for arches, so the span must contain butt joints. These joints represent discontinuities which must be accounted for. It will be shown later that the effective cross section is the minimum solid timber section which is at the section with the maximum butt joints. Figure 5.1 shows a typical SLT cross section.

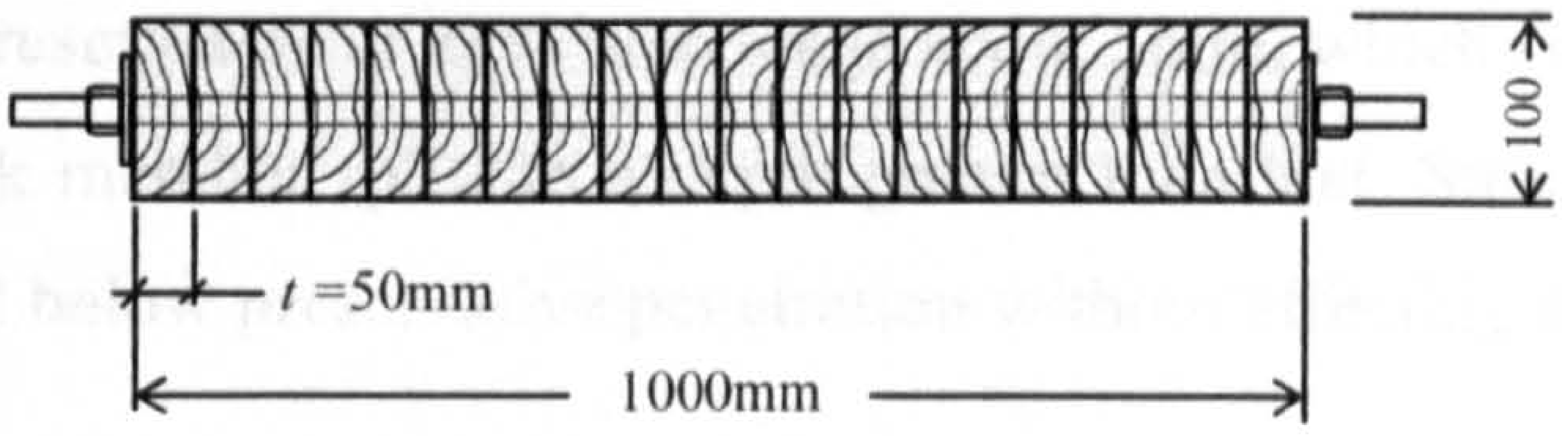


Fig 5.1 - Typical cross section of SLT deck

5.2 Material Properties - Timber

5.2.1 Timber

The main structural elements are softwood, and because this research is aimed at using home-grown timber, Sitka Spruce [49] is the first choice but Scots Pine and Douglas fir are excellent substitutes if available. Sitka Spruce was used for most laboratory tests. Mechanically stress graded timber was not used because the bending strength would not be critical at working loads in arches and it is very difficult to buy home grown stress graded timber of larger sizes. A visual stress grading system, based on the TRADA guidelines, was derived by the Author for use in the Forestry Commission and was used as the basis for grading the timbers in the structures – visual grading [Appendix 2]. This visual grading method is simplified by concentrating on growth rate and advising rejection of timbers with clusters of knots.

The moisture content (MC) of timber for construction is given in BS 5268 [50] as 18%. Eurocode 5 [51] specifies 16% for stress laminated bridge decks. The ambient moisture content will be approximately 18%, so this was specified for all untreated timber used in laboratory tests. All permanent bridges are pressure treated and the specification is given in Appendix 1. The specification states kiln drying to a 15% MC and then pressure treatment with Copper Chromium Arsenic (CCA). This low MC is specified to ensure a final MC, with treatment, of no more than 18% and also to ensure a greater penetration of preservative. BS 5268 [52] recommends a MC of below 28% but the Author has found the penetration of preservative at 28% to average about 5mm, which is unacceptable if an external timber deck member splits to a depth greater than that. Structural timber codes permit splitting well below preservative penetration without affecting the strength.

To ensure a design life of fifty years, the treatment process must be rigorous. Each laminate must be cut and drilled before treatment and the completed deck must be kept dry. An advantage of this type of structure over one using fewer, larger timbers is the effective treatment within the core structure. Pressure treatment with modern equipment never achieves full penetration, but treating each laminate before construction means that the resulting deck is more effectively fully penetrated.

The MC of timber for stress lamination is critical because if it is too wet the moisture will squeeze out over time and the stress in the bars will be lost. This means that newly treated timber cannot be used. It requires to be allowed to air dry over three weeks or be returned to the kiln after treatment.

The recommended treatment is specified as CCA but this was legislated out of normal use in June 2004 in order to come in line with EEC legislation (Council Directive 76/769/EEC). One of a number of derogations is that its use is still permitted for bridge decks but it is now almost impossible to find a treatment plant willing to use CCA. The situation with Creosote is similar, so a less toxic treatment - Copper Chromium Phosphate - is now used. This is also suitable for handrails, which must use a mild treatment because of the risk of children touching the rails and then licking their fingers.

The external laminates are made out of a sustainable hardwood. They typically have 100kN bearing onto a 200mm diameter plate giving a bearing stress of over 3 N/mm^2 perpendicular to the grain, which requires a minimum D40 hardwood. Home grown oak is usually used because it just satisfies the strength criteria and, more importantly, its source is easily traced.

5.2.2 Material Properties - Steel Stressing Bars

The transverse steel bars are critical to the stress lamination system. They must provide the force to mobilise friction in order to transfer the shear from the applied load. Their tension reduces over time through metal relaxation, which is additional to the shrinkage of timber laminates, but they must retain a minimum amount of force to perform their task. This is ensured by choosing a tensioning bar with a large strain and tensioning the bar to at least 2.5 times the required tension [41]. The greater the strain, the more movement that can be tolerated while some stress is retained. For that reason the highest tensile steel is the best choice for permanent structures. On the other hand, mild steel would be adequate for laboratory tests because the stress only requires to be maintained during the test loading procedure.

Durability of the bars is also a consideration for the permanent structures. The aim is to build bridges with a fifty year life. This can be achieved with the timber by treating each laminate before construction and waterproofing the deck to protect against exposure. To ensure the same life for the steel stressing bars against exposure, some protection is necessary. This can be provided in a number of ways:

- galvanising
- sheradising
- sleeving and greasing
- using stainless steel.

Additional to the attack from atmospheric agents, Copper Chromium Arsenic (CCA) preservative can accelerate corrosion of steel and some of the new formulae of preservative treatments could be more aggressive. For these reasons galvanised steel bars should be galvanised to a high specification, ensuring a thick coat of zinc. Sheradising is only useful for shorter life structures as the coating will only provide protection for a little more than five years. Sleaving and greasing is a useful alternative but it requires larger holes in the timber and, during construction, the sleeve must be positioned very accurately at the start because it cannot be moved later. This is the best way to avoid corrosion from the preservative treatment. Stainless steel bars are being used for the first time but are more expensive. They will be resistant to corrosion without sleaving but have different yield properties, which might require a different stressing regime.

All of the bars used in the laboratory tests were mild steel threaded bar. This allowed easy and accurate stressing up to tensions suitable for the tests. To ensure a high level of accuracy, a gig was built to convert torque from a large wrench into a known tension. A section of bar was connected at one end to a load cell in the frame with the other protruding past the frame and anchored with a hexagonal nut. This nut was tightened with the wrench, and torque settings were measured against load cell readings. Appendix 4 shows the results.

The bars used for all of the permanent bridges were supplied by Dywidag [28] who supply prestressing bars for ground anchors and concrete structures. Their standard product is a prestressing bar with a yield strength of 900/1100 N/mm^2. It is available in 15mm diameter and 20mm diameter, with a right hand thread. The bars are supplied either uncoated or sheradised or can be sleaved and greased. 15mm diameter bars were used for the first 6m span bridge without any protection and the second test bridge used 20mm diameter sleaved bars. Subsequent permanent bridges have used Gewi [53] galvanised bars which have a yield of 500/600 N/mm^2, Figure 5.2. Full technical data is provided in Appendix 5. They have proved easier for construction but the strain is reduced compared to the 900/1100 N/mm^2 prestressing bars and there is a question mark over the durability with the preservative treatment. Gewi bars are available in 16mm and

20mm diameter, with a left hand thread. To produce the initial prestress of $1N/mm^2$, 16mm diameter bars have been used for footbridges and 20mm diameter for vehicle bridges.

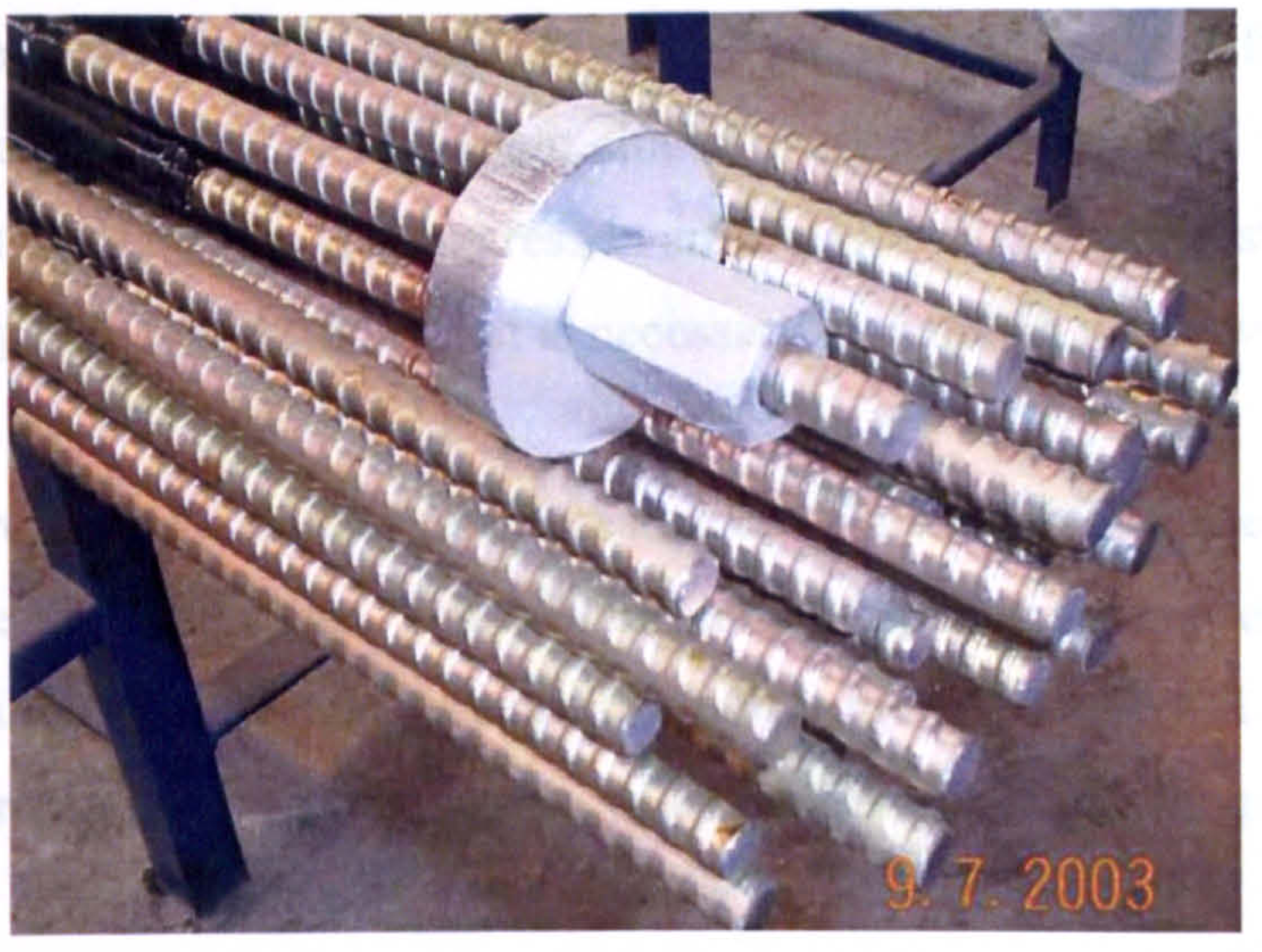


Fig 5.2 - Gewi stressing bars

As part of future work, consideration will be given to the use of prestressing cable [54] and glass reinforced plastic (GRP) [55] strand to tension bridges. Both have been used in the USA and the GRP strand does not relax. However, both are more sophisticated technologies and do not relate well to a low cost simple bridge system.

5.3 Prestress Effects – Basis of Design

Prestress is necessary for this structural system to disperse load laterally. An initial prestress of approximately 1 N/mm^2 is the target to provide enough friction in a flat deck [48] with sufficient allowance for stress loss, which is inevitable. The Ontario Highway Bridge Design Code (OHBDC) [56] and subsequently the AASHTO design procedures limited the steel to timber ratio so that losses due to creep could be minimised by maximising strain, while minimising strain loss. The steel to timber limitation specified in the OHBDC is typically 0.0016 and is species dependent. Subsequent Australian research by Crews – 1995 [57] concluded that a better way of achieving maximum bar tension to give maximum strain to counter creep was based on transverse bending stiffness of the plate. In the UK, as no specific research has been done with the native species, the bars are tensioned to 90% of yield to maximise strain. However this is not such a critical issue with arches, which act mainly in compression.

It will be shown that there is a major difference between arches and flat decks in the value of prestress necessary. An important aim of many of the load tests was to show the high load capacity at very low lateral stress in arches. However, there is still a lower limit and the following will demonstrate what is necessary to maintain that lower limit.

Because arches represent a smaller risk, threaded bar with a spring washer system will be tried in the future for small spans. This will be similar to the laboratory tensioning system and will make future maintenance easier. The nut at the fixed end will have to be tack welded to the bar to prevent the bar turning when maintenance tightening takes place.

5.3.1 Stress Relaxation

This has been well researched and documented in the USA, Canada and Australia and the results are transferable to UK practice and SLT arches. There was no reason, therefore, to carry out tests as part of this research. The relaxation is mostly due to creep of the timber laminates as a result of high stresses, perpendicular to the grain. There is also relaxation of stress in the steel bars which can amount to a 65% loss [58] through stretching over time though over short time-spans, this is a small component. Seasonal effects contribute to loss of prestress, so the effect is difficult to predict analytically. It is also dependent on moisture content and ambient temperature, as discussed below.

The AASHTO specification for these bridges [41] requires an initial prestress 2.5 times the minimum required tension to account for losses. Furthermore, the tendons must be re-tensioned to initial prestress level before leaving the site after construction, and again after eight weeks. Further restressing is advised at yearly intervals for the first three years.

5.3.2 Loss of Prestress – Creep, Moisture Content

Creep is the major component of loss and is inter-related with moisture content (MC). The creep mechanism in this case is a compressing of the fibres at right angles to the grain [59]. There is also a component from the surfaces of the laminates compressing together. Creep is time-dependent but with seasonal effects occurring simultaneously, evaluation and prediction is difficult. However, laboratory measurements were made by Crews in Sydney, Australia [48] and a 60% to 90% loss was observed in an eighty day period. Similar results have been observed in the USA [60] with 33% losses in nine days. The creep loss is species dependent [48] therefore it would be advantageous for some tests to be done on Sitka Spruce. However, tests have been done elsewhere on Douglas Fir and these are transferable. Crews also measured creep with different tensions and found that losses occur whatever the tension.

The conclusions on creep loss are, that if the laboratory tests under controlled humidity held true for permanent bridges in the field, they would not be practical because they would need restressed three or four times a year - Crews 2002 [48]. In practice, this has not been found necessary in the UK or elsewhere which shows that the laboratory effects are an accelerated scenario. Creep losses do not affect laboratory load tests. Creep losses fluctuate in real structures and are adequately allowed for in the AASHTO code by specifying a prestress 2.5 times the stress required.

Below the recommended moisture content of 18%, this effect is small as was observed in the laboratory tests, because the timbers were not treated and stressing times were short. Permanent bridge timbers, however, must be treated with preservatives and in some cases there was not always time to re-dry after treatment. When stressing took place in these cases, moisture and preservative was seen squeezing out and the bridge deck lost its prestress quickly, requiring restressing. This happened during the construction period on site when most of the excess fluid was expelled by restressing, before leaving site. In general, the restressing recommendations take care of this potential problem. Records are being kept when bridges are maintained so that a UK register of prestress loss can be compiled.

5.3.3 Loss of Prestress – Temperature

Work has been done in the USA [61] because continental extremes of temperature must be catered for. It was found that the losses were related to moisture content and very low temperatures. Significant losses in the bar tension were found where the MC of the timber laminates was above 30%. Where the MC was at the optimum of 18% the effect was small. When low temperatures returned to normal, tensions recovered. There is therefore little need to carry out work in the UK in this field.

5.3.4 Stressing Procedure and Tensions

Stressing procedures are well documented and it is essential that all bars are tensioned gradually. Too much tension too quickly resulted in cracking of laminates in one permanent bridge in the Forest of Dean. In all cases successful stressing was achieved by beginning with very low tension along the entire length of the bridge, until all the laminates were bedded together. Tension was increased to 50% of full tension on every second bar and then on the intermediate bars before repeating the procedure for full tension. Final tensions were 90% of yield to achieve the maximum safe strain and bar diameters were chosen to achieve a minimum of 1 N/mm^2 stress.

Laboratory tests followed the same stressing procedures. However, all tests included a measure of load capacity with different tensions which meant that, after full stressing had taken place, the tension was removed and the test tension then applied. This ensured bedding of the laminates, even for low tensions.

Although tensions are directly related to friction, which is the mechanism for transferring load laterally, it is also the mechanism for longitudinal transfer past butt joints. In arch bridges there is a locking effect when bars bear against the edge of the holes through which they pass. This provides additional resistance to the longitudinal shear and although this is not part of the design, it is a safety factor against collapse.

5.3.5 Shear between Laminates

The coefficient of friction between two pieces of timber depends on surface texture and moisture content. The value is between 0.2 and 0.5, Kempe's Engineering Handbook [62], the lower being wet and the higher dry with sawn finish. In the USA similar values were found - 0.3 for planed timber and 0.55 for sawn - and they were not dependent on species [63]. Initial prestress in the stressing bars is aimed to produce at least 1 N/mm^2 stress between softwood laminates which is much higher than is required to induce enough friction to transfer the shear forces. These values are however used to allow for stress relaxation and it is accepted that 0.34 N/mm^2 is enough to transmit wheel loads on a vehicle bridge [64]. The arch bridges under test in this thesis do not take point loads therefore friction is predominantly a factor to transmit horizontal shear past the butt joints.

To illustrate these points a series of tests on three 2.1m span and four 6m span laboratory bridges was conducted, with varying lateral tensions, and the results are detailed in Chapter 6 and 7.

5.3.6 Lap Lengths

This is one of the key differences between flat decks and arches. Arches require short laminates to form the shape, which results in more butt joints, and it will be seen in Chapter 6 that a typical layout is four holes per laminate of between 1m and 2m length. The holes are drilled on the radius curve so that, when assembled, the arch is automatically formed. The laminates are laid in a stagger pattern with butt joints, which leaves only 75% of the deck width solid at butt joints. Some bridges have only had 3 holes per laminate to reduce the number of stressing bars. This is now considered poor practice as it requires the cross sectional area to be reduced to 67% of the deck. These reduced sections are used in computer analysis calculations. The reduced cross section was found to give analysis results which matched test results and accorded with findings by Oliva [65] in 1990. This arrangement means that only very short lengths of laminate are provided for stress development around a joint in an arch. However butt joints are predominantly in compression in an arch, whereas they are in bending in a flat slab.

Bending creates deflection, which results in horizontal shear increasing as the effect approaches the abutments. Timber is very poor at taking horizontal shear and the effect should be limited.

Figure 5.3 shows a detail of a typical single laminate for bridge and Figure 5.4 shows the construction of a bridge over the river Forth where a single laminated is being placed.

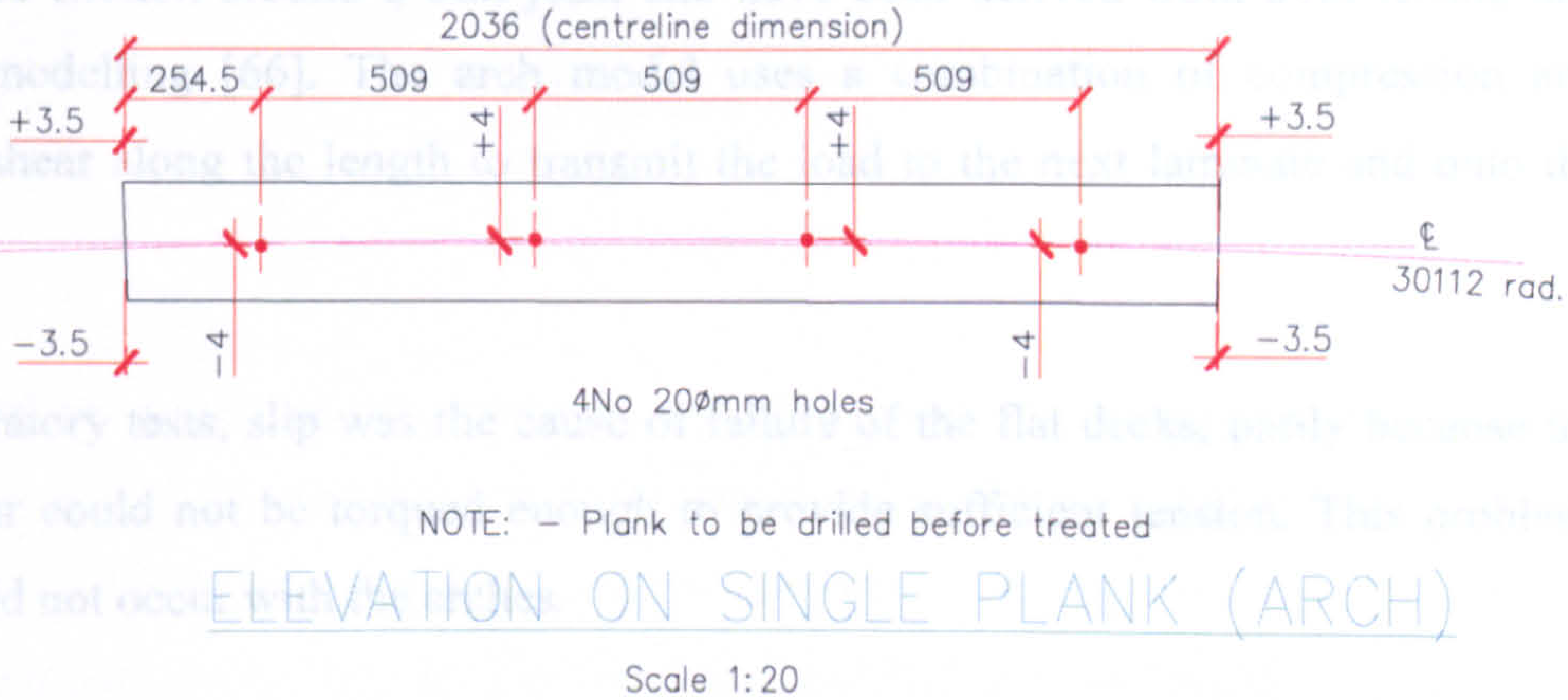


Fig 5.3 – Detail of a single laminate

Fig 5.4 Laminates on Forth bridge

The transfer length is the length between holes and it is the length which, through friction transfers the shear. This can never be more than 500mm for a large span arch and as little as 250mm in a 6m span arch. Butt joint spacings of 600mm c/c (with a 1 in 5 pattern), have been used for bridges in the United States, but spacings of less than 900mm c/c for softwood (1 in 4) pattern are not permitted in Australian design procedures. These limits are based on the minimum "development length" required to transfer load by interlaminate friction around a butt joint and have been derived from both testing and analytical modelling [66]. The arch model uses a combination of compression and horizontal shear along the length to transmit the load to the next laminate and onto the abutment.

In the laboratory tests, slip was the cause of failure of the flat decks, partly because the threaded bar could not be torqued enough to provide sufficient tension. This problem, however, did not occur with the arches.

5.3.7 Lateral Tensions and Lateral Distribution of Loads

Lateral tension in the prestressing bars is designed to create $1N/mm^2$ of stress between laminates, to mobilise friction for load transfer and to resist the prying forces from a point load. When a point load is applied, especially to a flat deck, the bottom of the laminates will tend to separate which means friction reduces and thus load transfer. This will especially affect lateral distribution of load. Arch structures will begin their structural action by first transferring load in compression and then in bending, resulting in smaller deflections. This will mean that any opening up on the tension face, or underside of the structure, will be reduced. This aspect of activity is not specifically measured as part of this project because footbridges are designed for UDLs. Detailed analyses were carried out by Crews 2002 [48] and the findings will be relevant to further stages of work in the UK. Figure 5.5 shows a detail of a deck opening up on the underside.

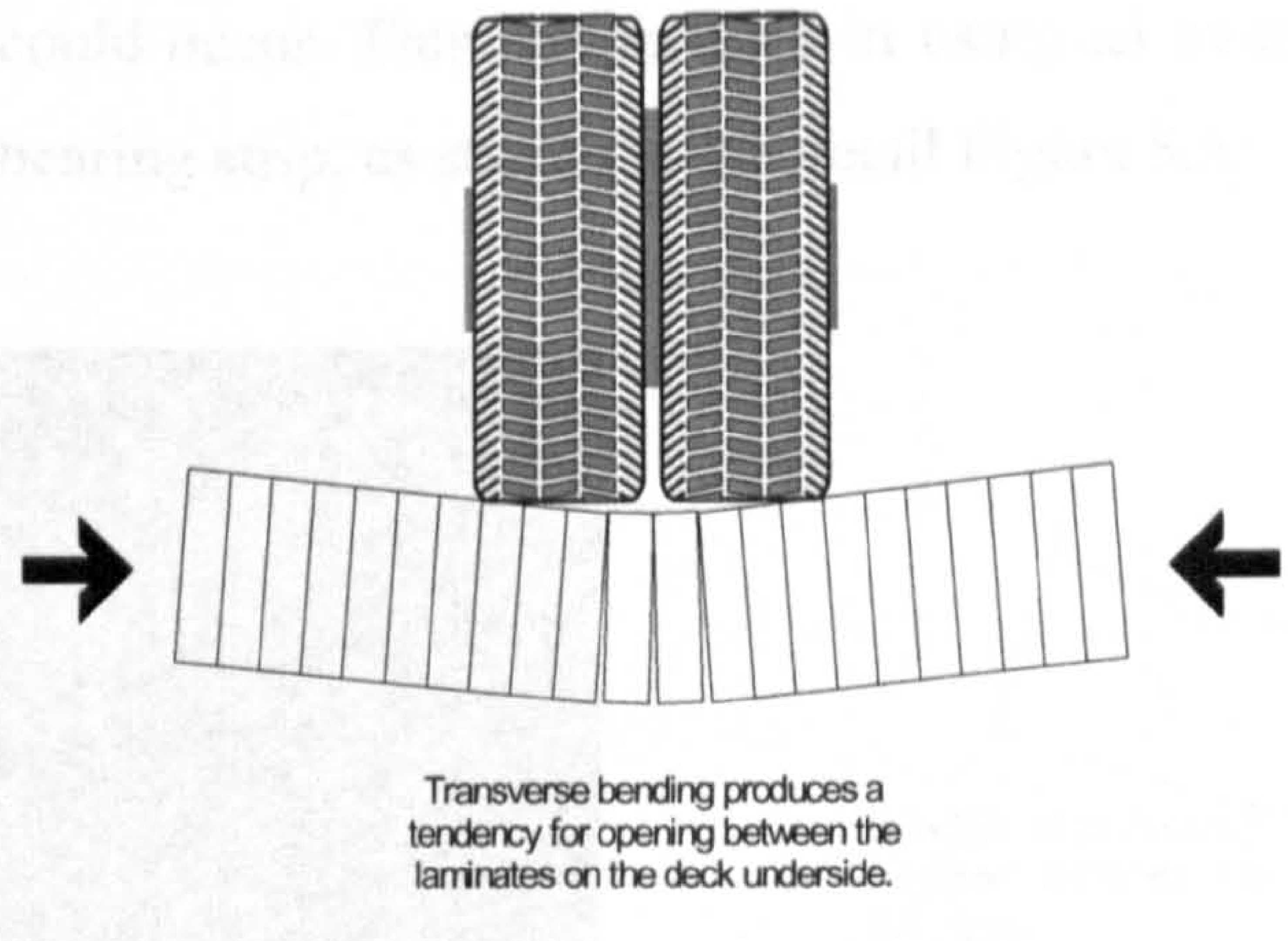


Fig 5.5 - Point load on deck, Crews 2002 [48]

It was found on a number of occasions that, when stressing a bridge, a bar could be tensioned to, say, 30kN and the nut at the other end of the bar was still loose. It was concluded that 100kN tension during construction was necessary on a full size structure to ensure even tension on the bar. The bar locks onto the sides of the holes in the timber and high friction forces have to be overcome. This is a very practical reason for tensioning bars to at least 2.5 times that required for load transfer.

This was not a problem with any of the laboratory tests because the structure was lightweight and did not create much resistance against the bar.

5.3.8 End Bearing Effects

The end bearings at the supports are a critical feature of the arch decks. The arches are of low rise profile so that pedestrians can easily walk over them and the maximum gradient permitted for all ability access is 1 in 10 [67]. This profile results in very high thrust loads at the abutments. This would be a problem for timber perpendicular to the grain but parallel to the grain, the forces are well within the permissible stress.

Drainage at this bearing is extremely important. The structure must have direct contact with the foundation but the detail must allow water which collects at that point, to drain away otherwise rot could occur. This has resulted in using an oversize galvanised steel channel and a small bearing strip, as shown in the detail Figure 5.6.

Fig 5.6 - End bearing detail - Carribber bridge

5.3.9 Anchorage of Stressing Bars

Permanent Bridges

This is the most highly stressed area of the bridge structure. The load must be spread into the timbers over the largest area possible to reduce the bearing pressure on the outside timbers. Achieving an interlaminar stress of $1N/mm^2$ and bars at, say, 500mm centres on a 200mm wide timber, requires a 100kN load, which is in turn spread over a maximum plate size of, say, 180mm diameter, giving a local stress of $3.9N/mm^2$. This load is perpendicular to the grain and so requires a minimum grade of timber of D40 [BS 5268 Table 8] [50]. This becomes a little more difficult for smaller bridges with laminate depth of 100mm. It is still general practice to use 16mm diameter stressing bars and they need to be fully tensioned to achieve the strain to maintain the friction. This means there is 100kN over, say, 100x100mm plate giving $10N/mm^2$ which is beyond all grades of timber. Normal practice has been to use the best sustainable hardwood available and accept some bearing failure.

On road bridges with larger, 20mm diameter bars, one attempt was made at using pieces of steel channel. This looked ugly and the thin web needed too much support. One answer is to use the maximum diameter spreader plate and the best hardwood and just accept some crushing. The spreader plates must be at least 20mm thick and supported by another similar but 80mm diameter to avoid unacceptable prying forces. A normal, full thread, hexagonal nut can be used with this anchorage.

The design of the anchorage is extremely important for maintenance. All of the permanent bridges have used 'Dywidag' threadbar or 'Gewi', a German derivative which is galvanised.

The DYWIDAG THREADBAR® Prestressing Steel has a continuous rolled-in pattern of thread-like deformations along its entire length. More durable than machined threads, the deformations allow anchorages and couplers to thread onto the threadbar at any point. The 20mm bar can be continuously cold threaded for its entire length, or supplied with threaded ends only. – see [Appendix 5].

The DYWIDAG System is primarily used for grouted construction. All components of the system are designed to be fully integrated for quick and simple field assembly. Sheathing, sheathing transitions, grout sleeves, and grout tubes all feature thread-type connections.

Threadbars are available in mill length to 18m, and may be cut to specified lengths before shipment to the job site. Where circumstances necessitate it, the threadbars may be shipped to the job site in mill lengths for field cutting with a portable friction or band saw. Threadbars may be coupled for ease of handling or to extend a previously stressed bar. Cold threaded 20mm diameter bars are available in lengths up to 7.2m.

These products are off-the-shelf and DSI [28] also provide the spreader plates. The secondary plate between the nut and spreader plate is 80mm diameter, sized to house the chair of the jack (white), Figure 5.7 which allows the nut to be tightened when the jack (red) is stressing the bar. If the bars have been cut short after initial stressing, as shown in Figure 5.7, an extension bar is joined using a sleeved socket. This is shown passing through the jack, and the back-nut (white), is threaded on to the extension, Figures 5.7 and 5.8.

Fig 5.7 - Tensioning jack & pump

Fig 5.8 - Tensioning jack

The jack is, unfortunately, extremely heavy because it needs to be robust in order to provide the 300kN thrust and very high pressure and also be capable of a stroke of about 100mm, which is necessary for construction. A maintenance jack, Figure 5.9, has different priorities. It must be as light as possible, be suitable for fixing through the timber rails and does not need such a long stroke. A system has been designed and tailored to suit so that it can be powered by the construction hand pump.

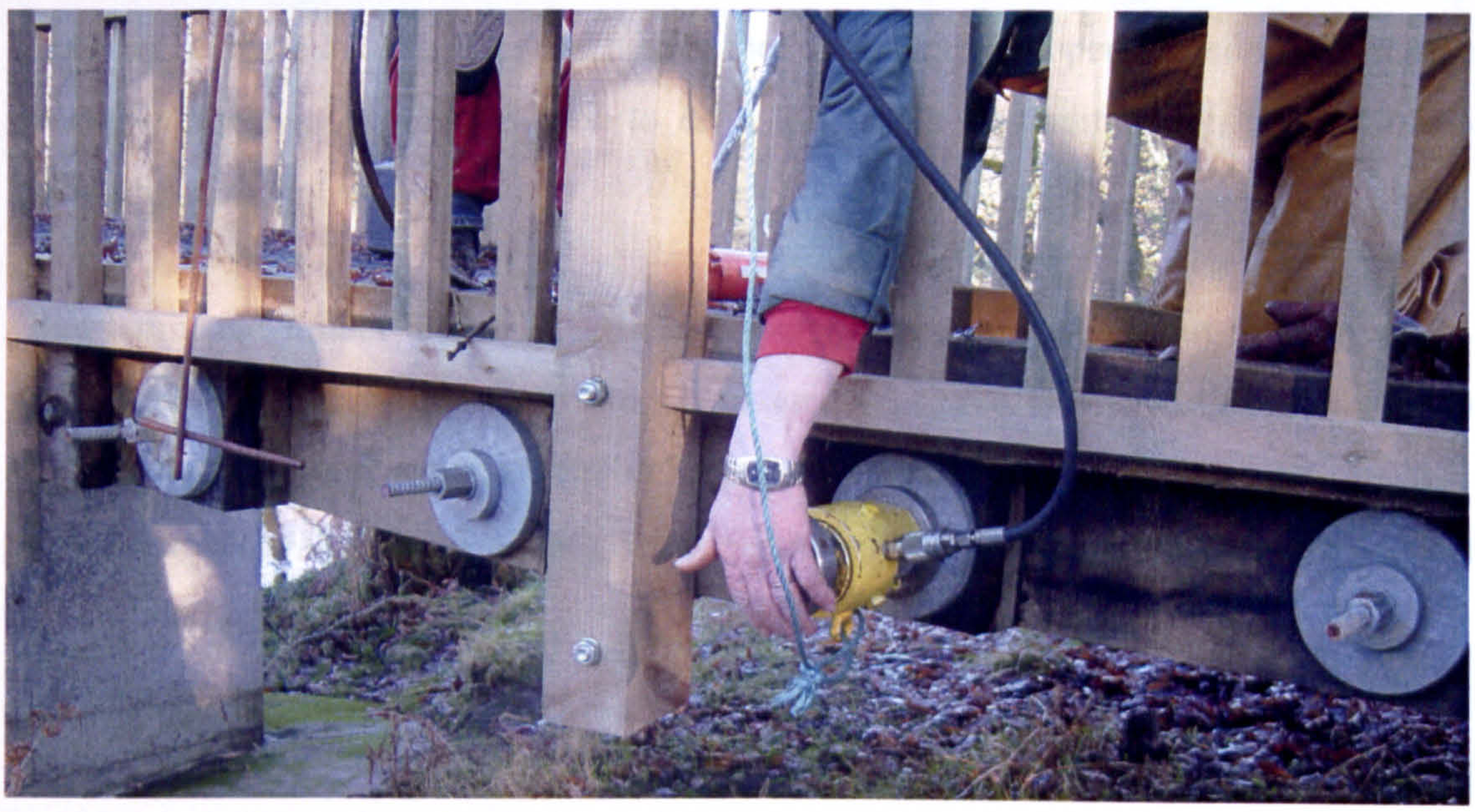


Fig 5.9 - Maintenance jack

5.3.10 Waterproofing of Decks.

The decks will always be pressure treated with preservative after cutting and drilling so should have a life of twenty years minimum, without maintenance. However this life can be extended by keeping rainwater off the deck, by waterproofing the surface. This would normally be done using good quality dense bitumen macadam which would contain a resin additive and 6mm whin stone aggregate chips. Asphalt can be used in a similar way quite successfully and will depend on availability. Clear mastic has recently been trialled on a mountain-bike bridge. This allows the natural timber to be seen through the coating.

Anti slip surfacing is becoming an important issue. A resin coating and dusting of bauxite chips over the bitmac provides an anti skid surface and improves the waterproofing. In 2005 a deck was covered with boards containing a fibre glass insert with resin and bauxite chippings, Figure 6.4. Trials are about to begin using green glues, normally used for finger jointing, sprinkled with bauxite chips.

5.4 Traditional Arch Analysis

Arches are automatically thought to be made of masonry and therefore it may be assumed, in error, that the structural mechanisms would be the same if an arch were made of another material. Timber and masonry are the original building materials but they could not be more diverse. They each have their own unique properties and it is a strange concept when one material is used to do a job which is, traditionally, thought of as belonging to the other.

Glue laminated timber arches have been popular over recent years but their structural actions are completely different to stress laminated structures. However, they do share the two important features which are timber's main structural assets -

- Timber is very good in both compression and tension and is moderate to good in bending capacity.

In contrast -

- Masonry is very good in compression but weak in tension; as a consequence it cannot be subjected to high bending stress.

However -

- Masonry is durable.
- Treated timber is less durable.

There is a market today, for short-term infrastructure, which timber is in a position to fulfil. A standard design life span of one hundred and twenty years for highway structures often results in expensive demolition as a result of planning or fashion not being well predicted for this number of years.

5.4.1 Masonry Arch Mechanisms

Masonry arches are, essentially, a balanced pile of stones. The ring comprises of two corbels supporting each other at the centre, with the fill and spandrel walls creating a counterweight against live loads. This balance is critical, as William Edwards found in 1746. He built a 42m masonry arch span over the Taff at Pontypridd four times, before he got it right. None of the failures were stress related. They were all due to the geometry of the arch. All early design was based on geometry. Figure 5.10 shows a secular arch (part of a circle), similar to the timber arches in this thesis.

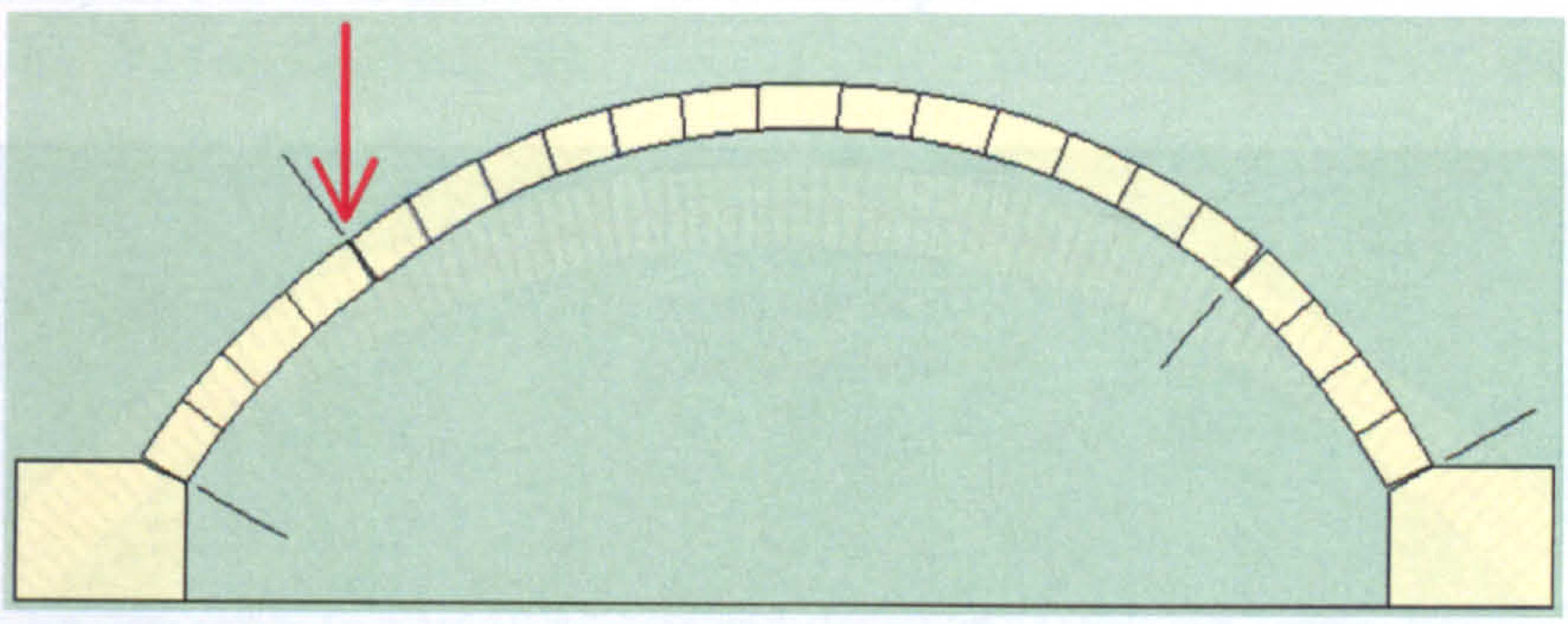


Fig 5.10 Secular arch [68]

It is shown with a load at, approximately, the quarter point, Figure 5.10, which is its weakest point, and it illustrates the formation of four hinges, two at the springings and two at quarter points. The thrust force which supports the applied load and the weight of the arch travels from one springing to the other, through the arch ring. As the applied force increases, the thrust moves up in the ring towards the load, to create equilibrium. If the applied force moves too far up, the hinge will open and the ring will go into tension. At this point the arch is unstable. Figure 5.11 shows the line of thrust at the edge of the arch ring which is the limit of stability.

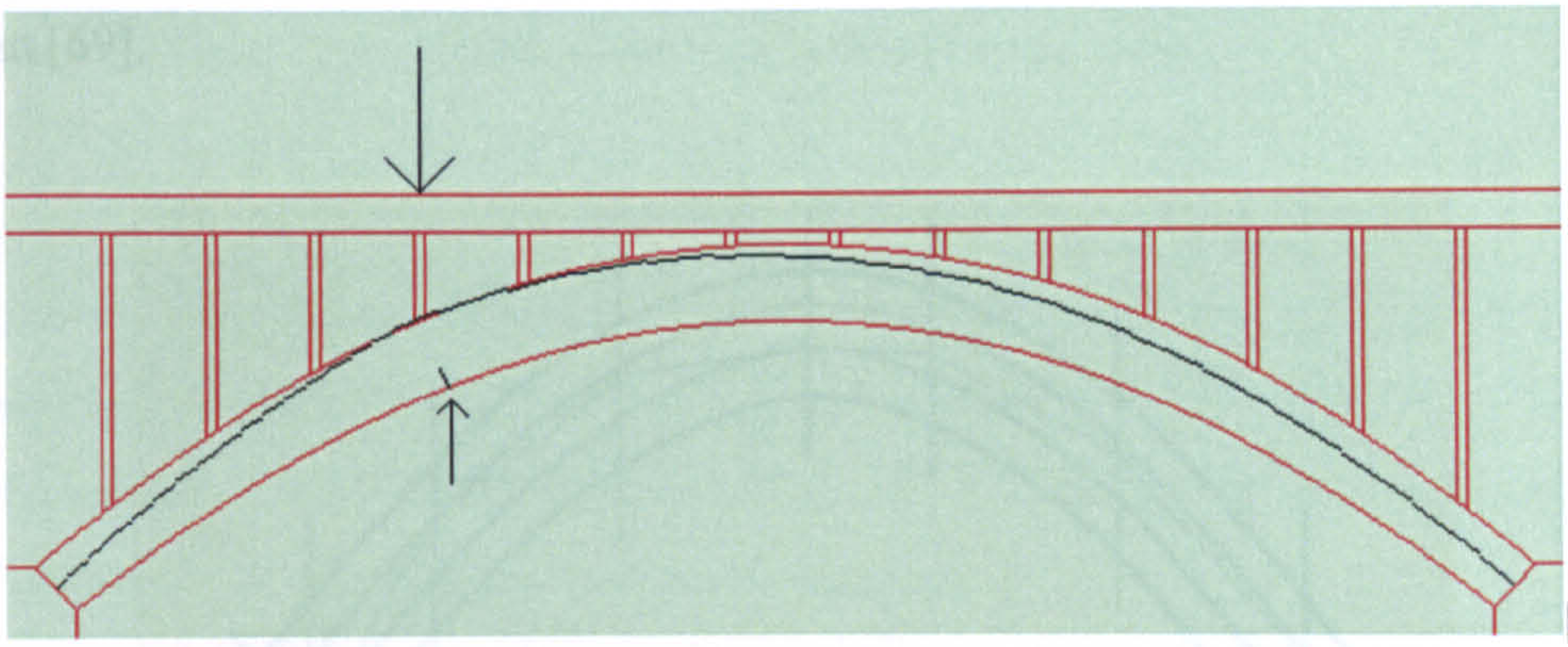


Fig 5.11- Arch thrust line

Figure 5.12 shows a funicular or catenary arch, which is the upturned shape of a piece of string hanging under no load. It is the perfect arch shape so the line of thrust, under uniform load, passes exactly though the centre of the arch ring. This makes it a very efficient shape to carry uniform load. It is very nearly parabolic which is easier to define mathematically so this is often used as an arch shape.

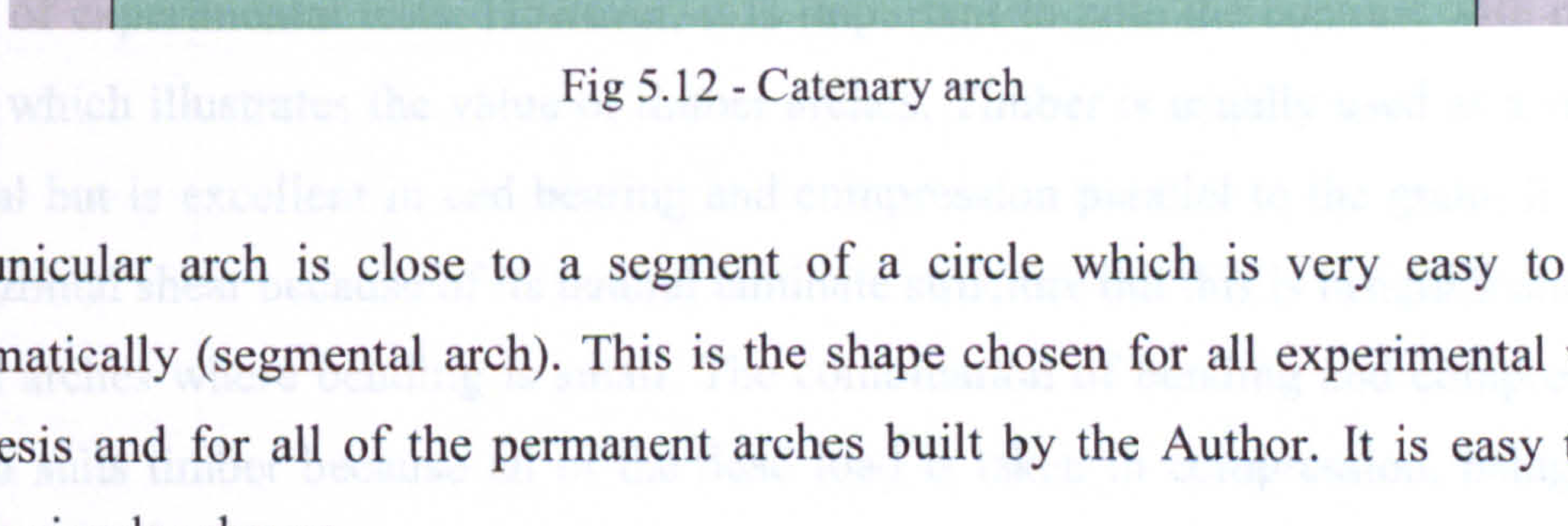


Fig 5.12 - Catenary arch

The funicular arch is close to a segment of a circle which is very easy to handle mathematically (segmental arch). This is the shape chosen for all experimental work in this thesis and for all of the permanent arches built by the Author. It is easy to build analyse circular shapes.

If two parallel lines are drawn to enclose the middle third of the arch ring, the zone for zero tension is enclosed, Figure 5.13. As live load is added to the arch, the thrust line move up and down and hinges are formed. If the line crosses the middle third line, tension begins. This was the basis of all early arch design and is described by Jacques Heyman [69].

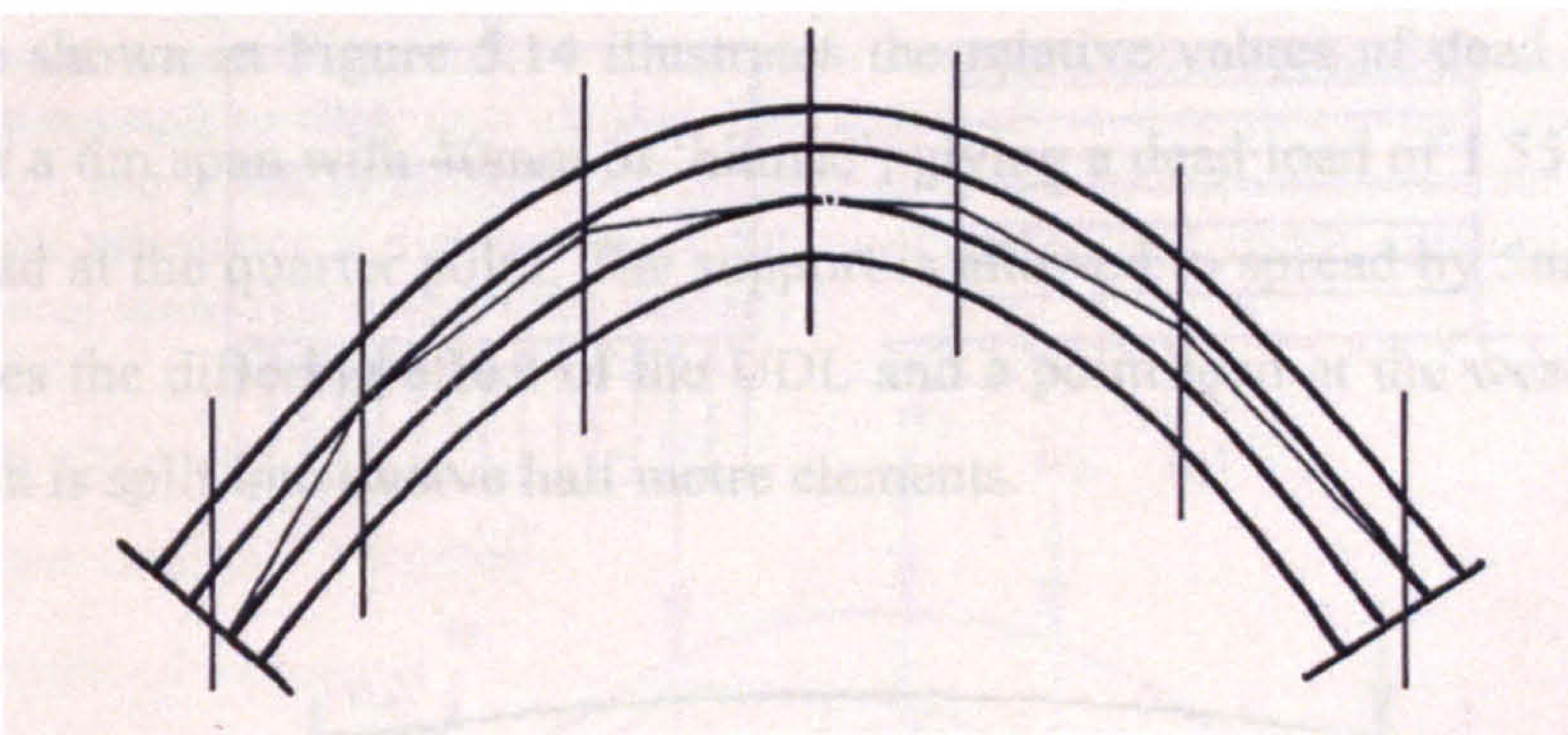


Fig 5.13 - Diagram showing arch ring with line of thrust in the middle third

This is the fundamental difference between masonry arches and those built from a ductile material. The masonry fails by forming a mechanism of hinges, leading to instability, and the timber arch acts elastically within its design limits. As greater loads are applied it changes shape dramatically but still takes increasing load in bending, until failure. Figure 5.15 shows a 6m span test arch near failure. The arch is inside out at the quarter point where the load is applied.

5.4.2 Timber Arch Analysis

In subsequent sections, the details of timber arch actions will be discussed with the results of experimental tests. However, it is important to note the contrast with masonry arches which illustrates the value of timber arches. Timber is usually used as a spanning material but is excellent in end bearing and compression parallel to the grain. It is weak in horizontal shear because of its natural laminate structure but this is insignificant in well shaped arches where bending is small. The combination of bending and compression in an arch suits timber because all of the dead load is taken in compression, being evenly distributed. The bending capacity is, therefore, reserved for live load.

The weakest sector of an arch is near the quarter point, therefore a live load at this position will produce the greatest bending moment. For that reason, all test arches were loaded at the quarter point as well as centrally. The central loading was applied as two line loads at ⅓ span and called four point loading. The full scale load tests were carried out using 10kN sand bags placed at the middle third of the span.

The example shown in Figure 5.14 illustrates the relative values of dead and live load deflection for a 6m span with 40mm of 'bitmac', giving a dead load of 1.55 kN/m^2, and a 20kN live load at the quarter point. The support is allowed to spread by 5mm, Table 5.1. This illustrates the differing effect of the UDL and a point load at the weak point of the arch. The arch is split into twelve half metre elements.

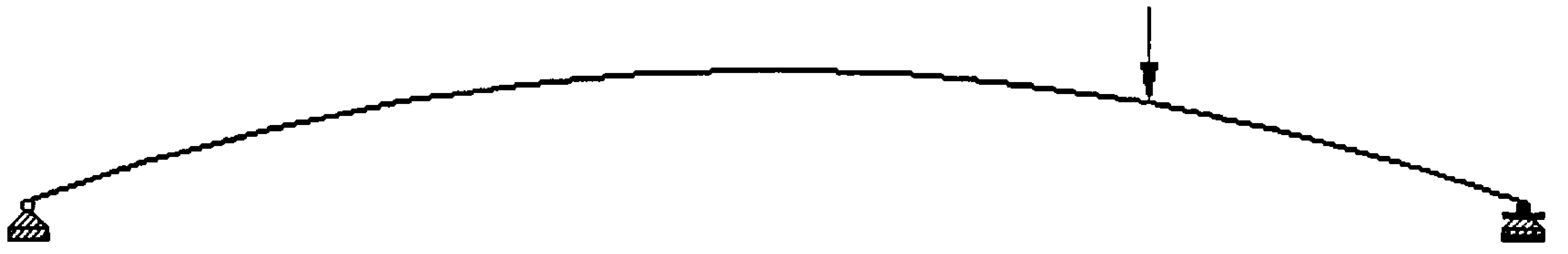


Fig 5.14 - Arch cross section

Table 5.1 Node displacements

Node	L/C	Z (mm)	Node	L/C	Z (mm)
1	DL1	0.000	8	DL1	-0.332
	LL1	0.000		LL1	-8.259
2	DL1	-0.111	9	DL1	-0.308
	LL1	4.646		LL1	-13.998
3	DL1	-0.199	10	DL1	-0.264
	LL1	7.979		LL1	-16.968
4	DL1	-0.264	11	DL1	-0.199
	LL1	9.130		LL1	-15.503
5	DL1	-0.308	12	DL1	-0.111
	LL1	7.731		LL1	-10.933
6	DL1	-0.332	13	DL1	0.000
	LL1	3.900		LL1	-5.000
7	DL1	-0.339			
	LL1	-1.781			

It can be seen in Table 5.1 that the deflections from a uniform dead load of $1.55kN/m^2$ are almost negligible, whereas the live load generated large deflection due to induced bending.

The design criteria for SLT arches will therefore depend on a number of variables:

1. span
2. rise
3. lateral tension
4. depth of section
5. timber grade
6. slip of laminates span
7. moisture content of timber
8. settlement of supports
9. loading

It is aimed to develop a semi-empirical model to simulate the performance (strength and stiffness) characteristics of SLT arch bridges incorporating the factors mentioned above and hence to compare them with the experimental results.

5.5 Behaviour of Statically Loaded SLT Arches

All of the timber arches that were tested displayed a high stiffness, when compared to flat decks. This was due to the geometry of the structure. Within elastic limits the bridges deflected very little so that, under normal design conditions, there is little risk of any semi- rigid topping used to waterproof the deck, breaking up due to severe change of shape. As loads were increased towards failure however, the arches were capable of very large deflections before any failure occurred. At high load levels the deflections were too large to be measured by the available transducers, and in order to avoid any damage they were removed before failure. Figure 5.15 shows the deflected shape of the first 6m span arch under a ¼ point load at what was termed failure. Under the load, the shape of the arch had reversed, causing the opposite side to take up a very tight profile. This caused one laminate to split along the grain through a tension bar hole, Figure 5.16.

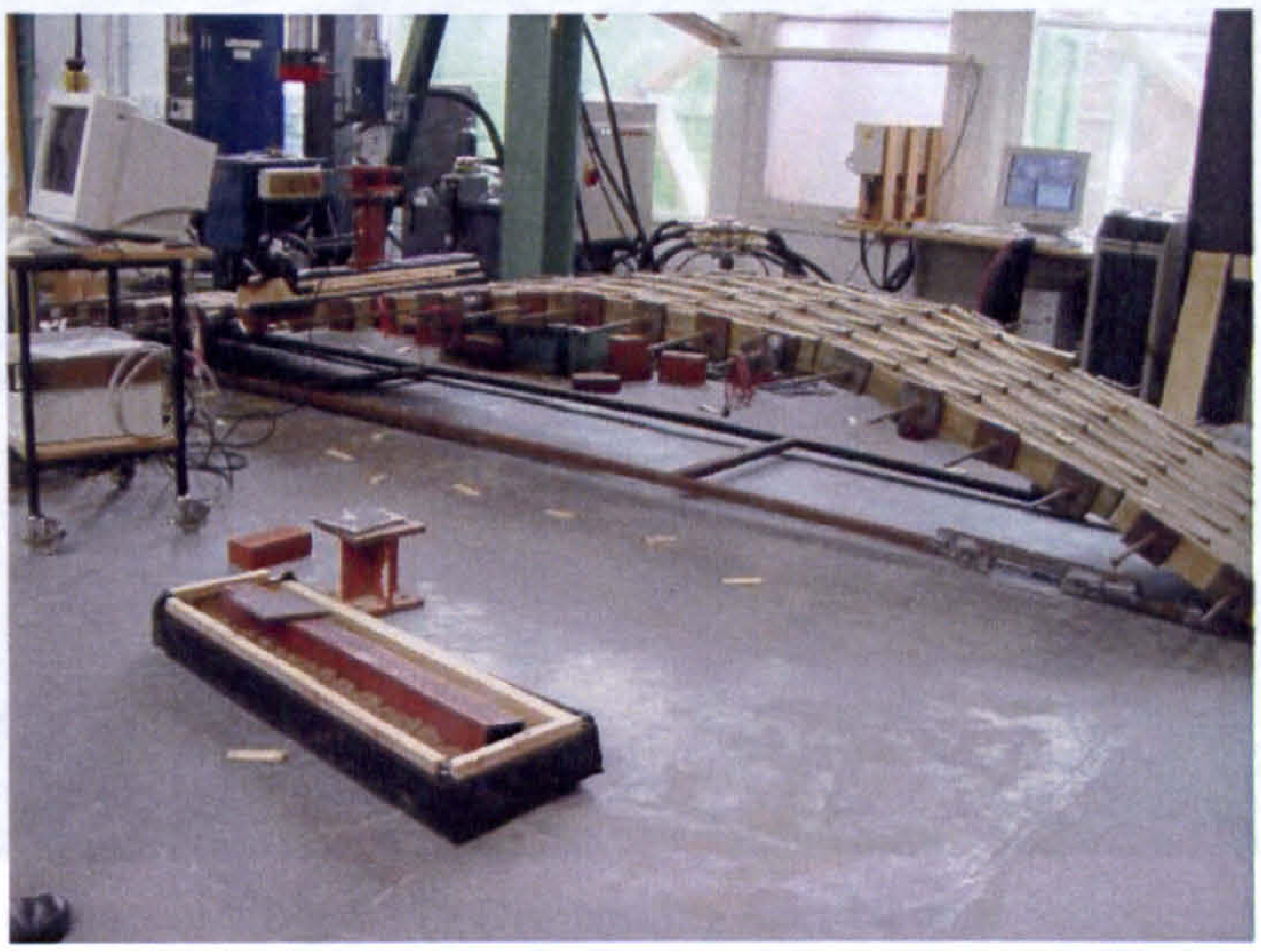


Fig 5.15 - Failure at severe deflection

Fig 5.16 - Failure – laminate split

When the arch was dismantled, Figure 5.17, the laminates were all undamaged except for one, which led to the conclusion that more load could have been applied before total collapse took place.

Fig 5.17 - Dismantling 6m trial arch

The bridge behaved linearly elastic with a line load of value equivalent to the design load of 3.2kN/m^2. With increase in load the excessive deformation of the arch lead to non-linear inelastic behaviour which no longer compared with the simple linear elastic results. Tests to failure were designed to determine the reserve of strength in the bridges and to give an indication of the safety factors involved.

The design load for the full scale test arches was 3.2kN/m^2 applied as a UDL. This type of loading is defined in the British Standard BS 5400 [23] which gives an intensity of 5kN/m^2. The reduction is justified because of rural location and a low probability of large crowds of people. The maximum eccentric load would therefore be half of the span loaded to one side of the bridge. This would produce approximately ¼ of the bending which caused failure in the 6m span bridge above, thus, a factor of safety of 4.

Vehicle loading on flat arch bridges will apply point loading at the quarter span and become the design criteria. However, vehicle bridges are more likely to consist of a flat deck supported by an arch therefore load will be, at worst, a point load at mid span. This is a strong point of an arch.

5.6 Behaviour of Dynamically Loaded SLT Arches

Arches are stiff when considering the minimal structural depth of section and compared to flat spans of the same section. They are rigid because their effective inertia is based on the square of the arch rise. However the arch ring itself is very slender and therefore could be quite active when a dynamic load is applied, although it may be strong in bending and compression.

There has been considerable activity over the last five years, since the Bankside Millennium footbridge over the river Thames was closed because greater than expected movements occurred, when a large group of people were crossing. The greatest problem was a result of the lateral forces induced by the pedestrians, which very quickly became synchronous, as they tried to maintain balance when the structure began to sway laterally. Another bridge, the Hungerford Millennium Bridge was under construction at the time and work was stopped while detailed analysis took place. Much of this work is published by Fletcher and Parker [70]. The important dynamic characteristics of a bridge are acceleration, Fundamental Natural Frequency (FNF) and damping ratio.

For non bridge-specific situations, BS 6841:1987 [71] gives the likely reaction of humans to root mean squared (RMS) acceleration as:

(a) a little uncomfortable: 0.3-0.6 m/s^2

(b) fairly uncomfortable: 0.5-1.0 m/s^2

(c) uncomfortable: 0.8-1.6 m/s^2.

BD 37/88 [24], ENV 1995 and Walther [72] quote acceptance levels up to $0.7m/s^2$ and Jiri Strasky [73] gives $0.5m/s^2$ for bridges with FNFs below 8Hz. He also states that maximum vertical acceleration $\leq 0.5\sqrt{FNF}$

The FNF of a simply supported flat deck can be calculated, as can a cantilever, but calculations are not reliable for an arch. In practice Engineers are now measuring dynamic characteristics of bridges after construction, to give accurate values before expensive damping is provided. It was therefore decided to measure the characteristics of the arches in this programme using three different sets of equipment which are described in Chapter 6.

When pedestrians walk, run or jump on a bridge they apply load and energy at specific frequencies. If the frequencies coincide with the FNF of the bridge, resonance will occur and if enough energy can be imparted, excessive deflections will result, which lead to high stresses. For simple structures, damping is not cost effective so the FNF should be large enough to avoid any chance of resonance. Strasky gives a very useful table, Table 5.2, to sum up pedestrians' pacing and jumping frequencies.

Table 5.2 – Pacing and jumping frequencies in Hz [73 – page 32]

	Total Range	Slow	Normal	Fast
Walking	1.4-2.4	1.4-1.7	1.7-2.2	2.2-2.4
Running	1.9-3.3	1.9-2.2	2.2-2.7	2.7-3.3
Jumping	1.3-3.4	1.3-1.9	1.9-3.0	3.0-3.4

The measurements of three of the 20m span bridges have given average FNFs at 3.5Hz before the topping was applied and 4.5Hz afterwards. The accelerations have all been low at approximately 0.02m/s^2. This shows that there is a possible problem with the most slender span but accelerations are low. This means that it is very unlikely that enough vandals could jump in sequence long enough to damage the bridge. This will be discussed in more detail in Sections 6.21 and 7.7. Figure 5.18 shows a very slender 20m span deck before handrails, which stiffen the structure, are added.

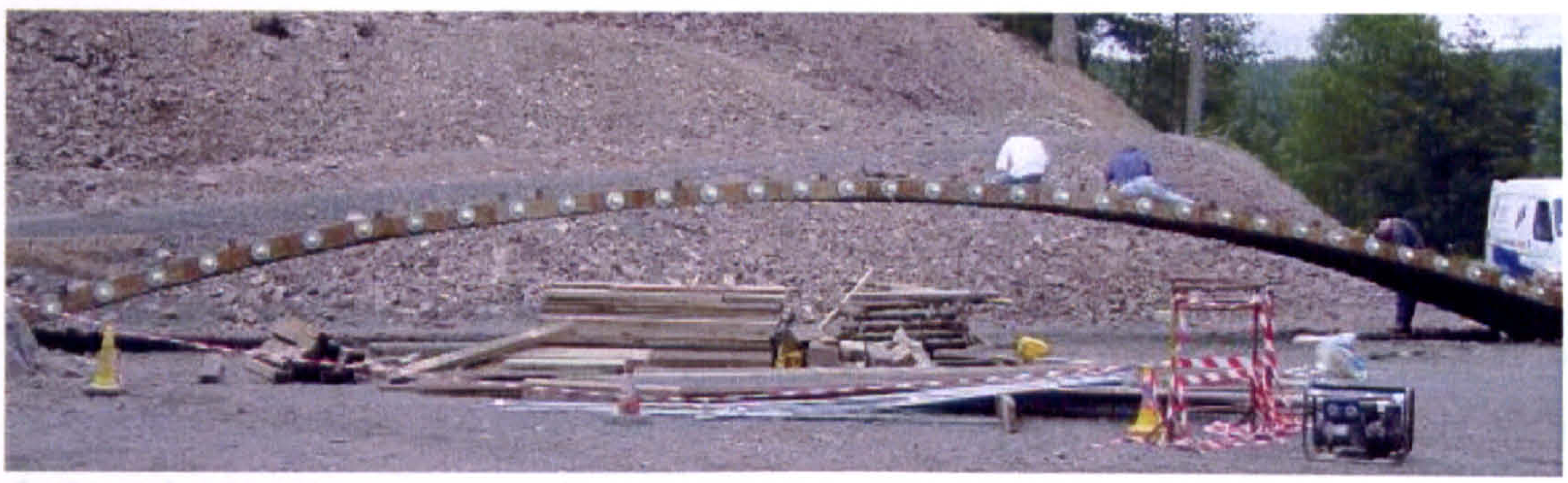


Fig 5.18 - 20m Test bridge at Glentress during construction – span/depth 1/100

5.7 Plate Models for Analysis of SLT Decks

Various analytical models have been used over the last twenty five years to provide deflections and moment distributions throughout a flat SLT deck. For practical engineering and design purposes, the beam analogy has proved the best approach:

- Orthotropic model
- Grillage model
- Finite element analysis
- Beam analogy

Orthotropic models were first tested at Queens University, Ontario, Batchelor et al – 1981 [74]. Later in the 1980s Forest Products Laboratory - Madison, carried out laboratory and some field tests to verify the Canadian test results. Variations of between 35% and 50% in transverse stiffness were found but the later work showed that analytical models of this kind could be used to predict design parameters, and this formed the basis of the AASHTO code. In 1993, in Australia, Crews [66] carried out full scale laboratory tests which validated orthotropic plate behaviour. However this was not considered a useful approach to design, as it is not easy to use the solution.

Work on grillage models for SLT flat decks has been based on concrete plates with limited success. This is a valid method but, again, mathematically intense for little benefit, Crews 2002, [48].

Some finite element modelling has taken place on SLT decks. It is a long process and only valid for the specific case which was modelled. This is important when working with timber, which is a very variable material. It has been considered that this intense method is more sophisticated than is necessary and useful.

The finding from the above on plate modelling for flat decks would be transferable to flat arches. For this reason no time was spent on sophisticated mathematical models, as no design advantage was perceived.

A simplified finite element analysis was used to calculate FNF of a 20m span bridge and produced a figure of 4Hz, being a check on the measurements which came to 3.5 Hz for the same decks.

5.8 Comparison of Composite Flat Decks to Arch Decks

Some earlier references were made to the restriction on spans for stress laminated decks because of the maximum size of timber available. This becomes a bigger problem with large vehicle loads. It has been tackled in Australia and the USA by building cellular and 'T' beam decks.

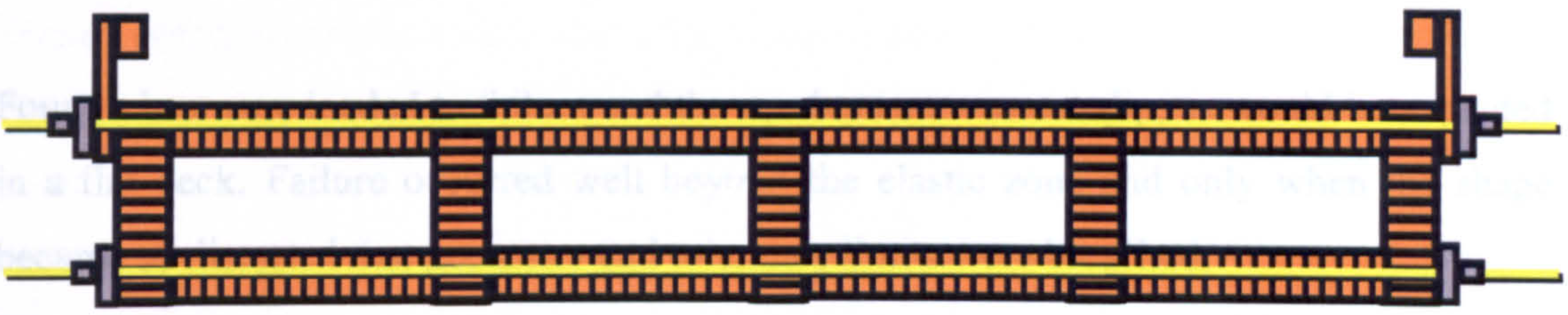


Fig 5.19 – Cellular deck – [75]

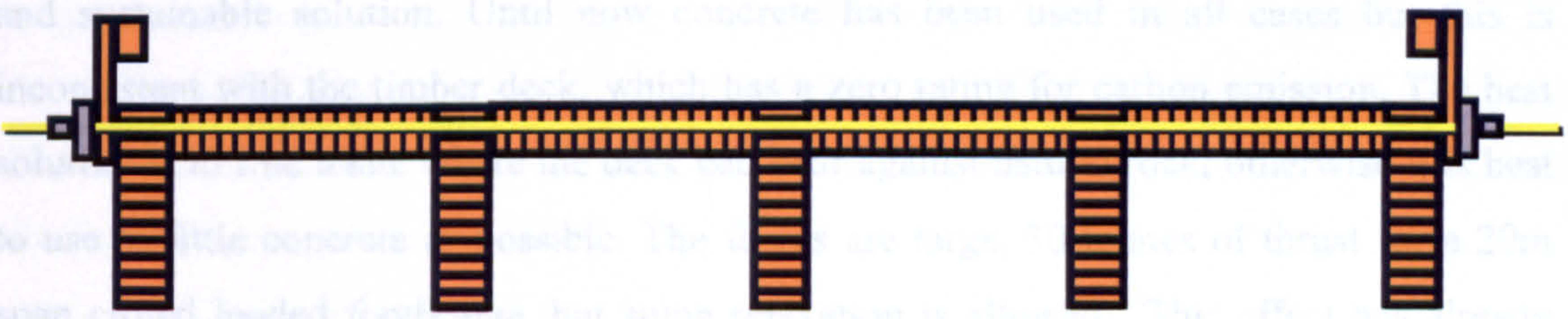


Fig 5.20 – T Beam deck – [75]

The decks which are shown in Figures 5.19 and 5.20 use SLT slabs in conjunction with LVL or Glulam beams to transmit horizontal shear. Unfortunately, as was stated earlier in this thesis, LVL is not available in the UK and closed boxes trap moist air or water, resulting in rot. These types of structure are suitable for warm dry condition but definitely not the UK.

In order to create successful longer spans for the UK market, it became obvious that any timber structures would require a system that did not use LVL and one that would dry in a free flowing wind. Arch structures are able to do that, and so this research project concentrated on the development of stress laminated arches.

The main reservation, before the first SLT arches were built, was the lap lengths and suitable load transfer. All of the previous work with flat decks specified longer lap lengths than would be possible in an arch. It was, however, proposed that the combination of compression and bending in the arch would equate to the greater bending of the flat deck. This has been proved to be the case in all arch tests.

Four arches were loaded to failure and the mechanism was not slip, as would be expected in a flat deck. Failure occurred well beyond the elastic zone and only when the shape became so distorted that geometry and tolerance limits were breached.

5.9 Foundations, Lateral Thrust and Settlement

The design of the abutments is an ongoing development of trials to find the most efficient and sustainable solution. Until now concrete has been used in all cases but this is inconsistent with the timber deck, which has a zero rating for carbon emission. The best solution is to find a site where the deck can bear against natural rock, otherwise it is best to use as little concrete as possible. The forces are large, 30 tonnes of thrust for a 20m span crowd loaded footbridge, but some relaxation is allowed. This effect has already been explored in laboratory tests where the lateral thrust was provided by tie bars which inevitably strained during the test. The increase in stress and deflection has been measured and is estimated to within acceptable limits, so this has been exploited in foundation design by reducing factors of safety. This movement was well illustrated during the test of the 20m span when 12 tonnes was placed centrally to measure deflections [79]. An 18m span bridge was built in Rochdale, England during Dec. 2005, using piled foundations to establish the cost benefit of this new approach to providing thrust at the springings.

Analysis calculations show a very large increase in vertical deflections if an abutment is allowed to settle, say, 25mm but the forces in the members change very little. Early measurements confirmed this with the 6m trial arch and the15m span, test-bridge, at the beginning of the research programme. The geometry of the arch shapes in all of the permanent bridges dictates that the vertical deflection caused by a lateral settlement will be 2.4 times the lateral movement. The deflection resulting from the load remains almost the same. If the span of the arch is increased by 25mm and the analysis re-run the increases in member force are very small. A vertical laminate arch of this kind settles into a new shape and takes the load without any significant increase in member load.

During construction the arch settles within the tolerances of holes and cuts. To avoid the final structure settling below its intended shape the shuttering should be set to add a pre-camber of about 1/200 times the arch span.

The method statement for the construction of these arches states that the horizontal foundation settlement and the construction settlement are to be assessed and the appropriate vertical pre-camber is to be built in to the shuttering. When all of the laminates are in place and tensioning begins, the supports are removed and the arch is allowed to settle which also pushes the foundations out. The tensioning is then resumed. This avoids any unnecessary locked-in stress and allows some passive resistance behind the foundations to be mobilised.

5.10 Maintenance

The success of SLT arches will depend on how much maintenance is required. With considered actions, all of the negative factors surrounding the durability of timber arches can be designed out. The biggest problems are:

- rot of timber
- maintaining the lateral tension
- stiffness of foundations.

Rot is designed out by adding preservative treatment under pressure and then keeping the treated timber dry during its working life. The covered timber bridges in Europe and the USA have lasted more than one hundred years, Appendix 6. The roofs were provided to shed snow and in fact kept the bridges dry, which is why they are still in good condition. Roofs are not practical today because of the height of lorries, so it was decided to cover the deck with a waterproof, dense bitumen macadam. Pictures in Appendix 6 show metal plates used to shed rainwater from SLT decks, in Australia and Norway.

Lateral tension is essential to live load sharing between the laminates. Bars are chosen at the design stage to be stressed to 90% yield while imparting $1N/mm^2$ between the laminates. This level of stress guarantees a Factor of Safety of 2.5 to 3 in the lateral tension because $0.3N/mm^2$ is sufficient for friction. At this stage of development in the UK, it is considered that re-tensioning is required at the end of the construction period, then eight weeks later and again after one year. Subsequent checks will be recommended at five year intervals thereafter.

A number of studies have been carried out on residual tension as part of the U.S. Forest Service's Timber Bridge Initiative [46]. Taylor. S reported on eighteen bridges [75] which were checked for lateral tension, moisture content and bearing damage in 1997. All of the bridges had been in place for many years and had not had regular maintenance. The conclusions were that slip of some laminates had allowed moisture into the structure which affects slip and load capacity. The important conclusion is shown in Table 5.3, that only about 10% of tension bars were found to have fallen below the 40% value of initial tension. This is encouraging and shows that this form of construction is reliable in practical terms.

Because this aspect of stress lamination is so vital and so much useful work has been carried out by others, a small bibliography is included, on Page 263.

Table 5.3 - Summary data for stressing bar forces of eighteen stress-laminated T-beam and Box-beam bridges inspected in West Virginia. Data in this table were sorted by thickness of the flanges.

Statistical Quantity	Bridges with 178 mm Thick Flanges	Bridges with 229 mm Thick Flanges
Number of Bars Tested	35	52
Mean (kN) [kip]	67.2 [15.1]	57.8 [13.0]
Median (kN) [kip]	67.6 [15.2]	57.8 [13.0]
Coefficient of Variation (%)	28.0	30.3
Maximum (kN) [kips]	96.5 [21.7]	105.9 [23.8]
Minimum (kN) [kips]	25.8 [5.8]	16.0 [3.6]
Initial Design Bar Force Level (kN) [kips]	74.7 [16.8]	96.0 [21.6]
Allowable Bar Force Level (kN) [kips] (40% of Initial Design Level)	29.8 [6.7]	38.2 [8.6]
Number of Bars at Stresses Below Allowable Level	3	6
Percentage of Bars Tested Below Allowable Level (%)	9	12

Recent tests on completed bridges in the UK have shown encouraging results.

A 17m span arch at Golspie, which was built in Spring 2005, did not receive its eight-week re-tensioning, as planned. It was eventually carried out in December 2005 and the majority of the bars still had between 30-35kN of force from an initial prestress of 90kN. The structure compressed between 4 and 5mm on re-tensioning. The 20m span Forth bridge was re-tensioned at the same time, four months after completion and the results were similar. It was found that the bars had to be tensioned from both sides of the bridge to achieve uniform results. Figure 5.21 shows the re-tensioning jack in its various stages of operation.

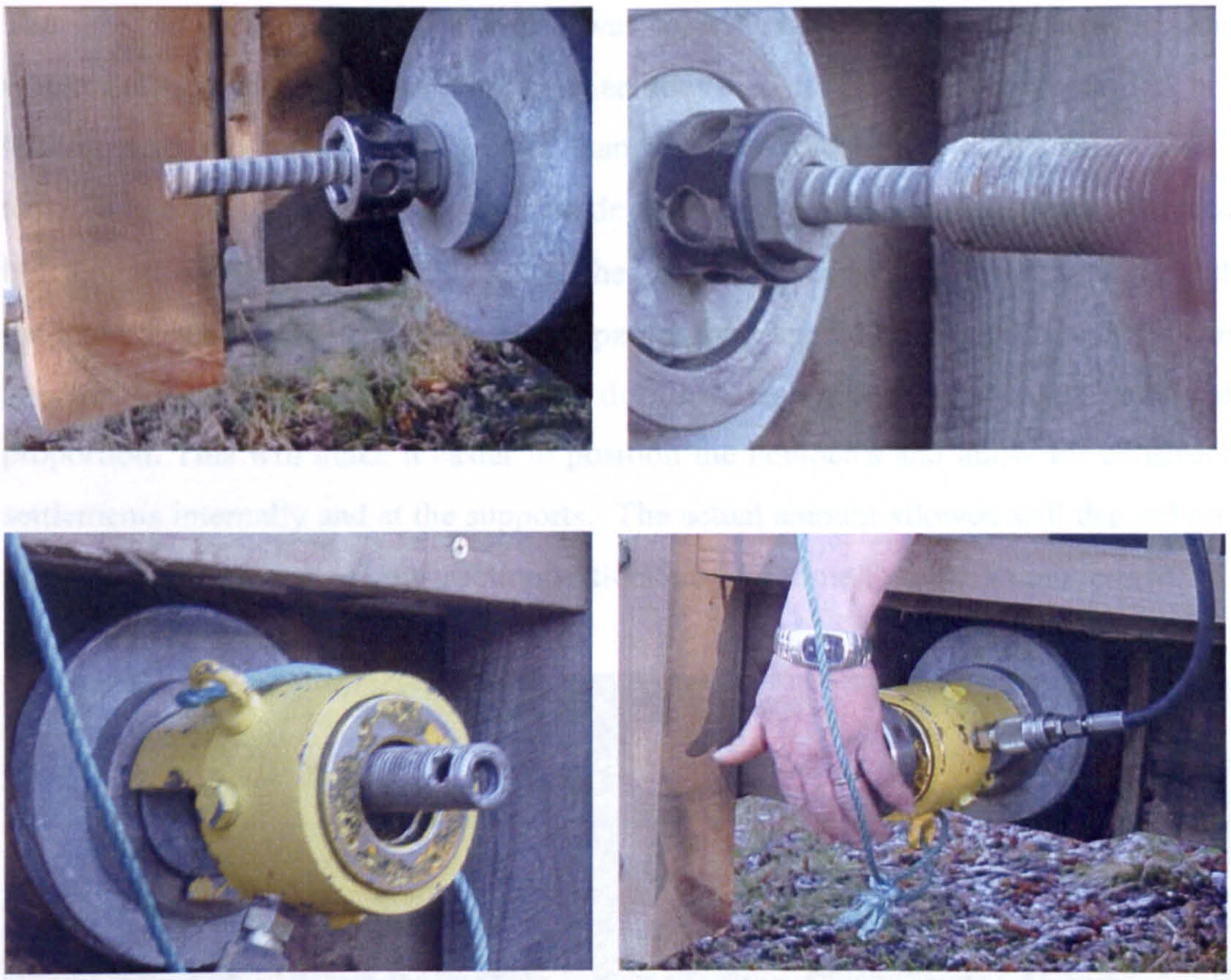


Fig 5.21 - Lightweight re-tensioning jack

5.11 Practical Aspects of SLT Arch Construction

Footbridges are very often situated in remote locations, which could, in the future, provide many opportunities for these bridges. All of the components are small and easily transported. Short spans only require quite primitive temporary works.

Detailing is important for durability and to create a tight fitting, end bearing arch which will take maximum load with minimum slip and settlement. Construction techniques must be as safe, effective and as low cost as possible. This is a new bridge type so these aspects had to be as well thought out as possible. Many lessons were learned from mistakes made, as work progressed.

The most important detailing exercise was to ensure tolerances were to a maximum, where they could be, and minimum when they had to be. Holes were drilled in the laminates to a diameter 3mm greater than the outside diameter of the threads on the tensioning bars. Angle end cuts were made at dead size as the laminates are designed to butt against each other. The tolerance in the holes is the construction tolerance for sliding the laminates onto the bars. The central span support should be set approximately one or two hundredths of the span higher than the final rise, with quarter points increased in proportion. This will make it easier to position the laminates and allow for construction settlements internally and at the supports. The actual amount allowed will depend on the Engineer's assessment of construction settlements and some foundation movement.

Fig 5.22 – In situ construction of a 20m span bridge

The most important construction aspect has been temporary support. Small span bridges up to 10m span are relatively light and are easily constructed on the river bank before being lifted into place. However, long spans of 20m can weigh 10 tonnes, which then require a large crane and close tolerance temporary supports. Figure 5.22 shows a 20m span at an early construction stage. It has become clear that long spans must be erected in situ. This necessitates a full scaffold, at a cost of £5-7k. Figure 5.23 shows a typical scaffold for a long span bridge.

Fig 5.23 – Full scaffold

The 15m span test bridge, built in the University car park, was self-loaded onto a lorry, Figure 6.33. It was transported 250 miles and offloaded onto a tractor and trailer and a rear bogie. It was then dragged to site and winched into place. This caused the bridge to move and stretch, which presented difficulties in achieving accurate final dimensions.

The careful setup of temporary supports is known to be vitally important. In two cases, the channel supports on the abutments were put in place at too shallow an angle, resulting in a non secular shape. This had an exponential effect on the natural frequency and stiffness. Quite minor excitation caused collapse of a 20m exhibition span, Figure 5.24. This unscheduled test showed the importance of the arch shape and was considered in subsequent dynamic tests. The Bridge shown in Figure 5.25 was erected without the correct rise at the centre and it also collapsed, this time due to vandals, reportedly, jumping at the reduced FNF of the flat arch.

Fig 5.24 – Collapse at RHS resulting from lack of shape

Fig 5.25 – Mallard Pike, Forest of Dean - completed too flat and collapsed

When all of the laminates are in place, the deck must be tensioned gradually and evenly because it can, very easily, twist in a horizontal direction. If this is allowed to happen, it cannot be rectified unless all tension is removed and the process begun again. Although the abutment bearing channels must be set exactly to the correct angle and level, even more important is that they must be squared by checking diagonals exactly. If one side has a span, say, 30mm longer than the other, the rise will be 70mm less, which makes the bridge very lop sided. The initial tensioning will be relaxed and the support removed to allow settling and end bearing to take place. The tensioning process must then be repeated gradually up to full tension. When the bridge is complete, usually about three weeks after first tensioning, it will again be re-tensioned and finally after eight weeks in service. As a safety check they will be re-tensioned again after one year until there is full confidence in the technique.

The moisture content of the timbers at the time of treatment and construction is of vital importance. The British Standard for preservative treatment [52] only requires drying to 28% before treatment which results in peripheral impregnation only and is too wet for stressing. The moisture content must be reduced to preferably 14% and then treated. The timbers will then be left to air dry, or again kiln dried back to 14%, if possible, before construction. If this is not done it is very easy to overload scaffolding and tension will be lost very quickly, as excess moisture is pushed out under pressure.

These very shallow decks have caused a problem with handrail fixing. The small structural depth needs to be increased by steel brackets to provide sufficient lever arm to resist lateral handrail loads. These brackets must be fixed well onto the span or they will overstress the external laminates. Figure 5.26 shows typical fixings which have proved successful. They still look close to the outer laminate but are very secure due to the stressing.

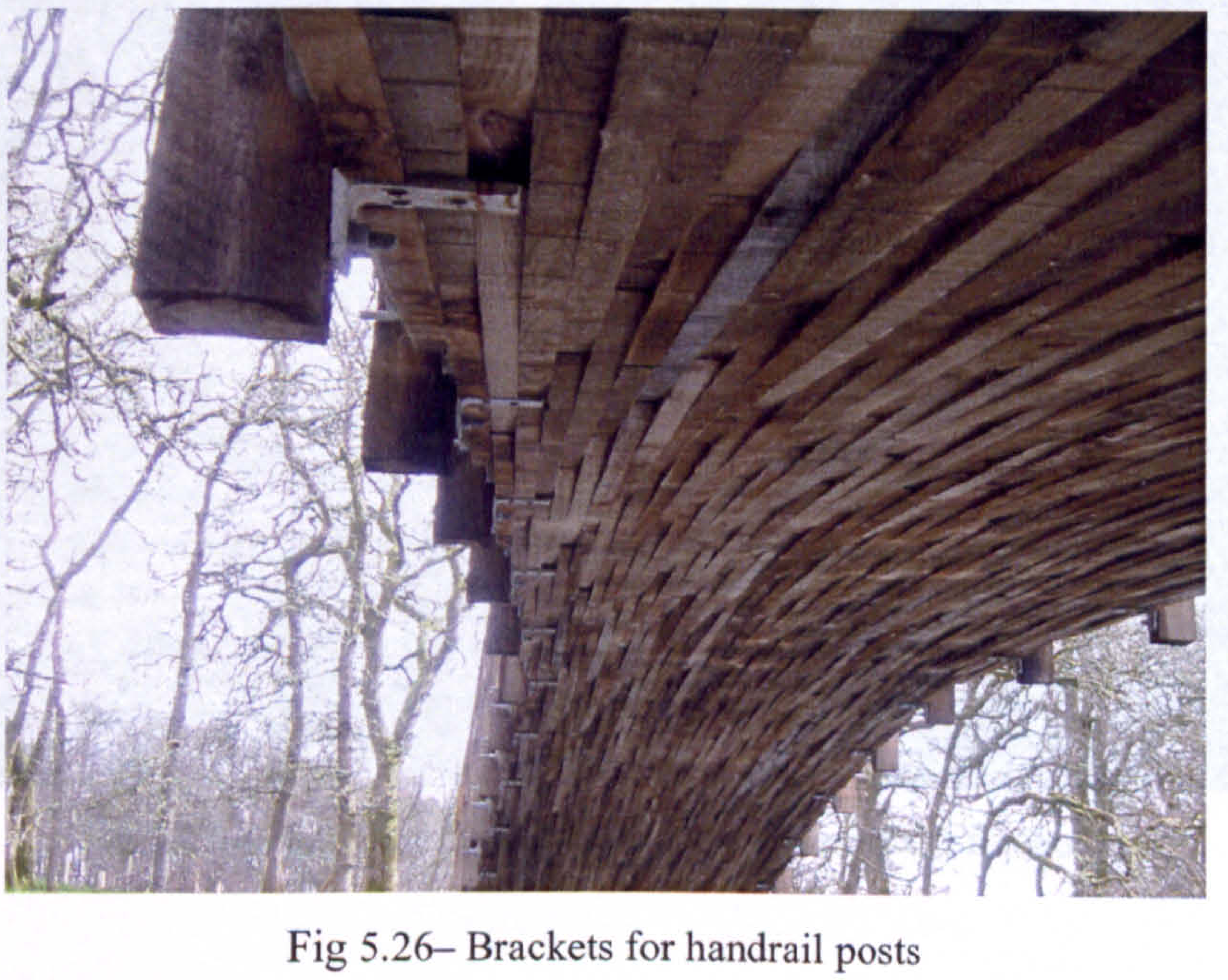


Fig 5.26– Brackets for handrail posts

The first SLT decks in the last century were nail laminated. This can be a useful technique for making the building of these, more structurally advanced bridges, easier. If laminates are nailed as the arch develops it makes stressing simpler because there will be less slack to jack up. This can however interfere with arch relaxation before stressing. It would be interesting to try very small nail laminated arches for use in remote locations. It would also be of value to develop the use of mild steel threaded bar for stressing small arches.

A final practical aspect of arch bridges is that they provide excellent freeboard for floating debris in a flood. The springings must be kept at least above the one in one hundred year flood level, which means that a medium sized bridge will have 1m of space for debris rather than the normal 300mm which is presently provided on flat span bridges, in order to minimise cost. Figure 5.27 illustrates these details well.

Fig 5.27 – 20m span over river Forth with excellent freeboard.

CHAPTER 6

6 EXPERIMENTAL WORK

This chapter deals specifically with the arches chosen for experimentation and the testing procedures undertaken. It describes the apparatus used for testing and the specific construction details of the arches built for testing. It closes by describing each of the test bridges and their trial loadings in a chronological order and gives an overview of the results. This is accomplished by taking extracts from published papers by the Author, where appropriate. This leads on to Chapter 7, where all of the results are shown and analysed in detail, leading to the development of a generic semi-empirical model in Chapter 8.

6.1 General Considerations and Geometry

The first trial, SLT, bridge in the UK, was put together for the Royal Highland Show (RHS) in the year 2000. It was a flat deck of 3m span using Macalloy bars to stress it laterally. This displayed the concept and generated some interest. This bridge is now in a permanent location just inside the M25, near London, Figure 6.1. It taught the Author that the tension bars must strain a significant amount to maintain tension and 25mm Macalloy HYS bars did not perform that task well.

Fig 6.1 - First SLT bridge – Macalloy bars

The first SLT arch was the winning design for a competition, organised by Innovative Timber Engineering in the Countryside (InTEC) in the UK in 2002. The remit was to build an attractive and innovative timber footbridge. The design used the structural arch as the bridge deck. The span was 12m and an arbitrary rise of 1m was chosen to provide a reasonably stiff arch, while, at the same time, not being too steep to walk over, Figure 6.2.

Fig 6.2 - 12m span bridge

All commercial pedestrian arches, built since then, have had the same span/rise ratio. Two sets of test arches built in the laboratory have had different ratios in order that the effect on stiffness and strength could be measured.

The slope, up to the quarter point on a 12 to 1 ratio arch, is too great for wheelchairs (> 1 in 10) so fill has been placed to flatten the slope. The decks have all been waterproofed with dense bitumen macadam, containing a polymer additive. A 40mm layer is bonded to a dry deck with a tack coat and is thickened at the springing to reduce the slope to 1 in 7, which is still a little steep for wheelchairs but it is over a very short distance. Details of this end section are shown in Figure 6.3.

Fig 6.3 - Details of end of deck and tar

Recently a pedestrian bridge was decked, using boards with non-slip inserts placed on fillets, which reduced the slope. The laminates were not waterproofed, although they were pressure treated with preservative, Figure 6.4.

Fig 6.4 – Non slip deck on new bridges at Salcey

Future road bridges will be a combination of flat SLT decks, integrated with arch supports. These structures will have higher natural frequencies and greater stiffness due to the connections between the arch and the flat. This will be the subject of a further study. A pedestrian bridge was built to this design as a prototype and is shown in Figure 6.5. The connections for a vehicle bridge will be more substantial with particular attention to the joints which may be integrated with the stress lamination of the decks. This would provide a complete structure with material of the same modulus resulting in even stress distribution and avoiding high local concentrations at bolt connections.

Fig 6.5 – Dalby bridge – arch with flat deck

A test programme was chosen to experiment with a variety of spans of the 12 to 1 ratio, both in the laboratory and in the field. A few flatter and steeper spans were tested in the laboratory to help build a full picture of the effects of arch rise on strength.

6.2 Test Apparatus, Instrumentation for Deflection and Force Measurements

The laboratory bridges were tested using a 'Dartec Modular 9500' which is a combined loading and data acquisition system with a 300 kN load capacity, Figure 6.6. This provided a measured load through hydraulic rams and stops at the preset test load or deflection. The loads were transmitted to the structures through a series of beams and rams and in order to ensure that the line load was applied evenly, a bed of sand was provided in a box on the deck.

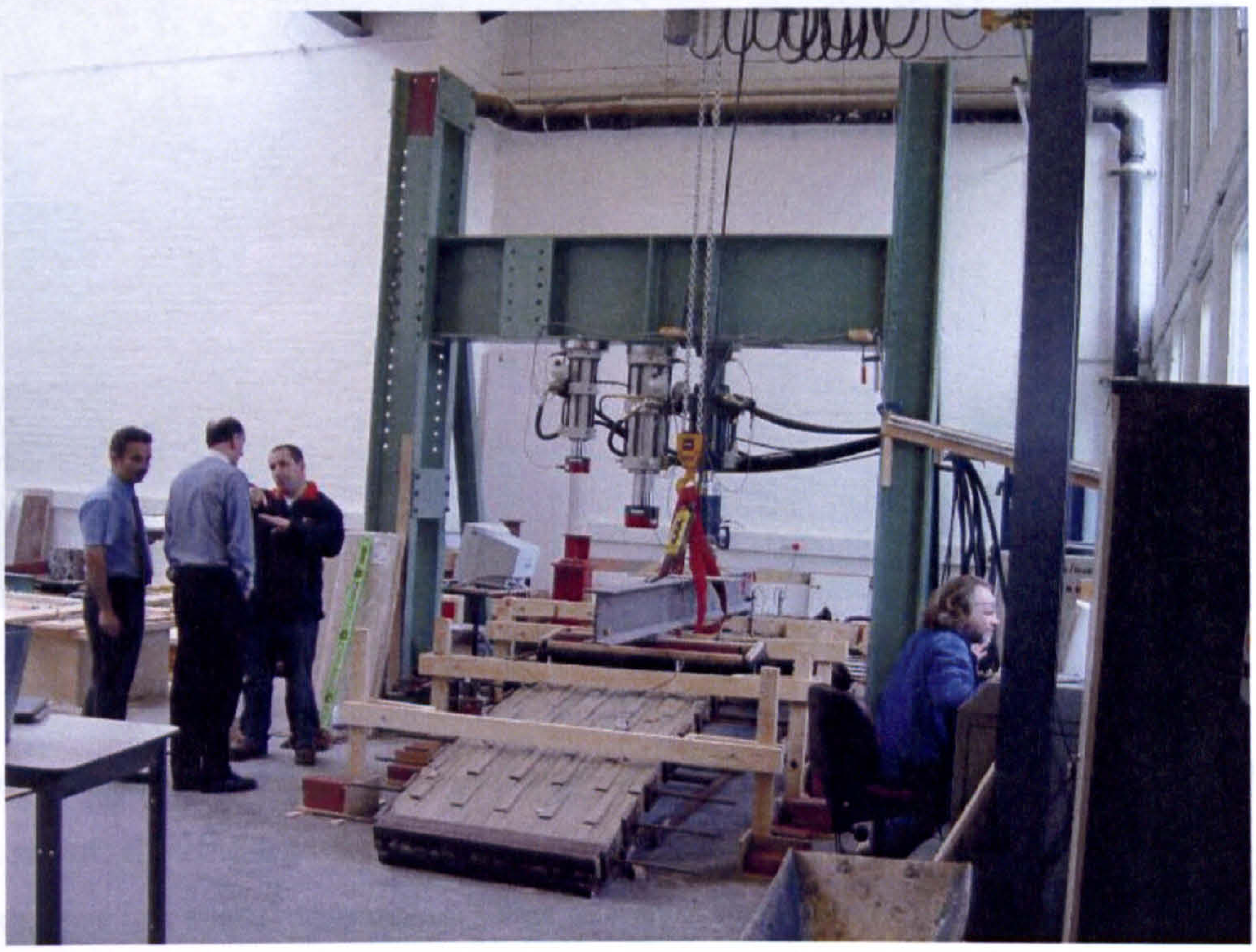


Fig 6.6 – Laboratory test of timber arch

Vertical deflections of the span and horizontal deflections of the supports were measured using displacement transducers with 100mm of measurement range. All outputs were recorded by an onboard data logging system and then analysed using Excel.

In the field, kentledge was used as loading which is not ideal, but the only practical solution. Loading with kentledge deviates from a laboratory load system, in that when the load effect is measured, the load continues to thrust on the deflected form. The laboratory rams, in contrast, stop at the position and load that a reading is required, thus allowing time to record. The most convenient form of kentledge was 10kN sand bags delivered by a building supply merchant. These bags are accurately weighed and the delivery vehicle was able to load and unload the structure.

Fig 6.7 – 20m span bridge with kentledge loading

The deflections of the full scale structures were large, compared to the laboratory tests, which meant that the travel of the transducers created a problem. Some had to be reset because their full scale measurement was less than the deflection, Figures 6.7 and 6.8.

Fig 6.8 – Transducer – tests at Glentress

Smaller movements in the thrust tie bars of the full scale bridges were measured using Wheatstone bridge strain gauges. The strains were converted to thrust. The smaller laboratory bridge tie thrusts were measured directly using load cells, Figure 6.9.

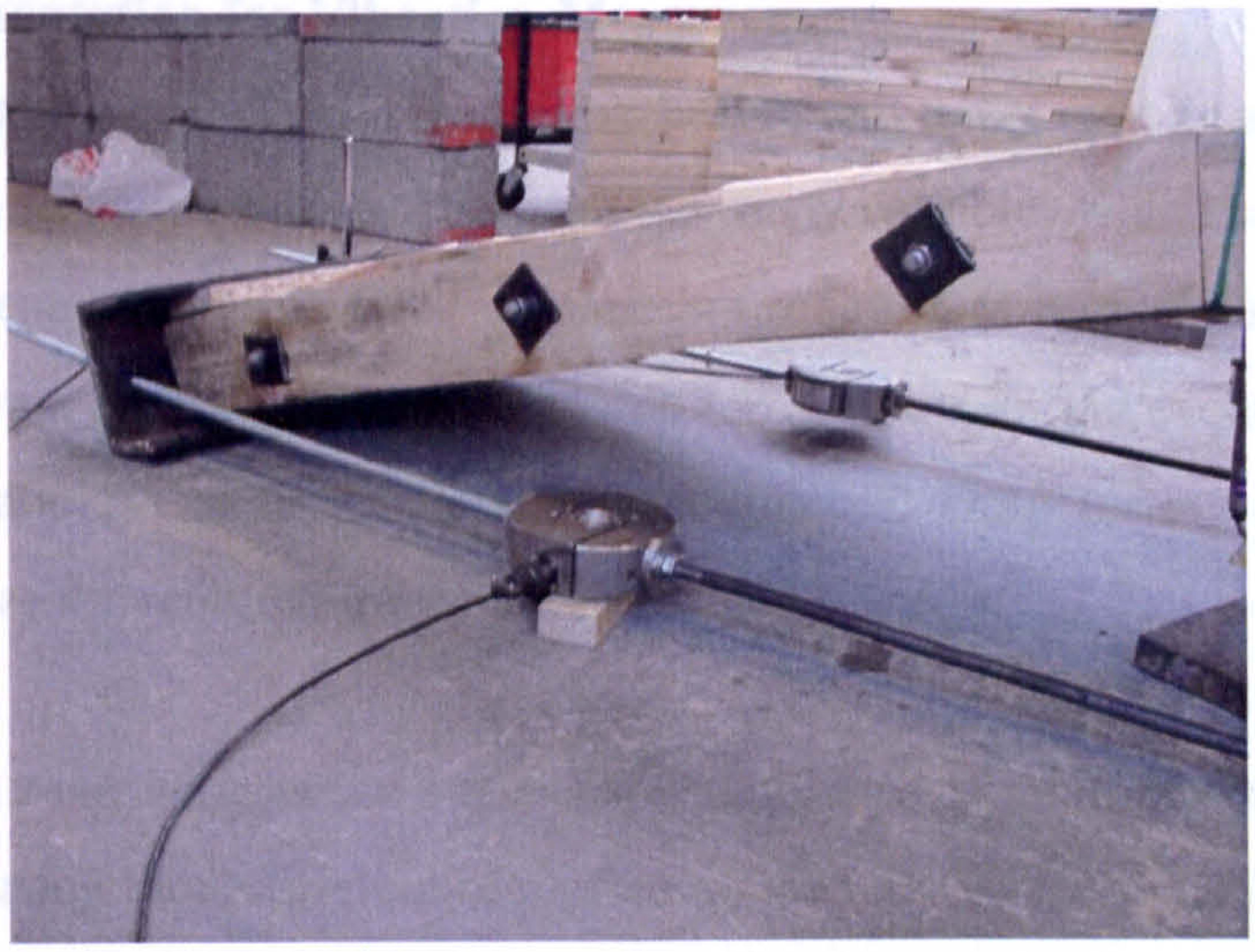


Fig 6.9 – Load cell

When the 20m span bridge was re-loaded four months after initial testing, the deflections were measured remotely, using a SOKIA 413R3 Total Station which measured any point in three dimensions to an accuracy of 1mm, Figure 6.10. This was the only opportunity in the entire test programme to measure longitudinal and lateral movement, as well as vertical. The instrument stored all of the readings for subsequent download directly to PC. This was very successful and saved on many of the site safety procedures, normally necessary for on-site load tests and is to be recommended for the future.

Fig 6.10 – Sokkia Total Station – Remote deflection measurements

6.2.1 Instrumentation for Vibration Measurement

The Natural Frequency (NF) of a laboratory bridge will never give an accurate indication of the NF of the full scale bridge. There are too many variables which cannot be modelled within experimental limits. In Civil engineering today, it is normal for large footbridge contracts to have a contingency sum set aside for dampers, in the event that they may be needed. Calculations cannot predict the specification. Only testing of the completed structure can give the required information.

Each of the full scale bridges were tested for a measure of their natural frequency (NF). This was done using three different sets of equipment, to ensure a check on this sensitive parameter.

The first set of equipment used four vertical and two horizontal Pinocchio Vibraphones, Figure 6.12, connected to an 8-channel TEAC LX10 data recorder, in conjunction with ARTeMIS test planner and modal analyses software. The excitation required was provided by two then four people, walking steadily over the bridge, Figure 6.11.

Fig 6.11 – Excitation for ARTeMIS test

In ARTeMIS Analyser, the data was processed with a default signal processing configuration, including a 1024-lines spectral density estimation. During the measurements modal analyses were made using the Fast Frequency Domain

Decomposition Peak Picking technique. This was for quality checking of the data as well as verification of the sensors and their positions. Figure 6.12 shows an accelerometer in place.

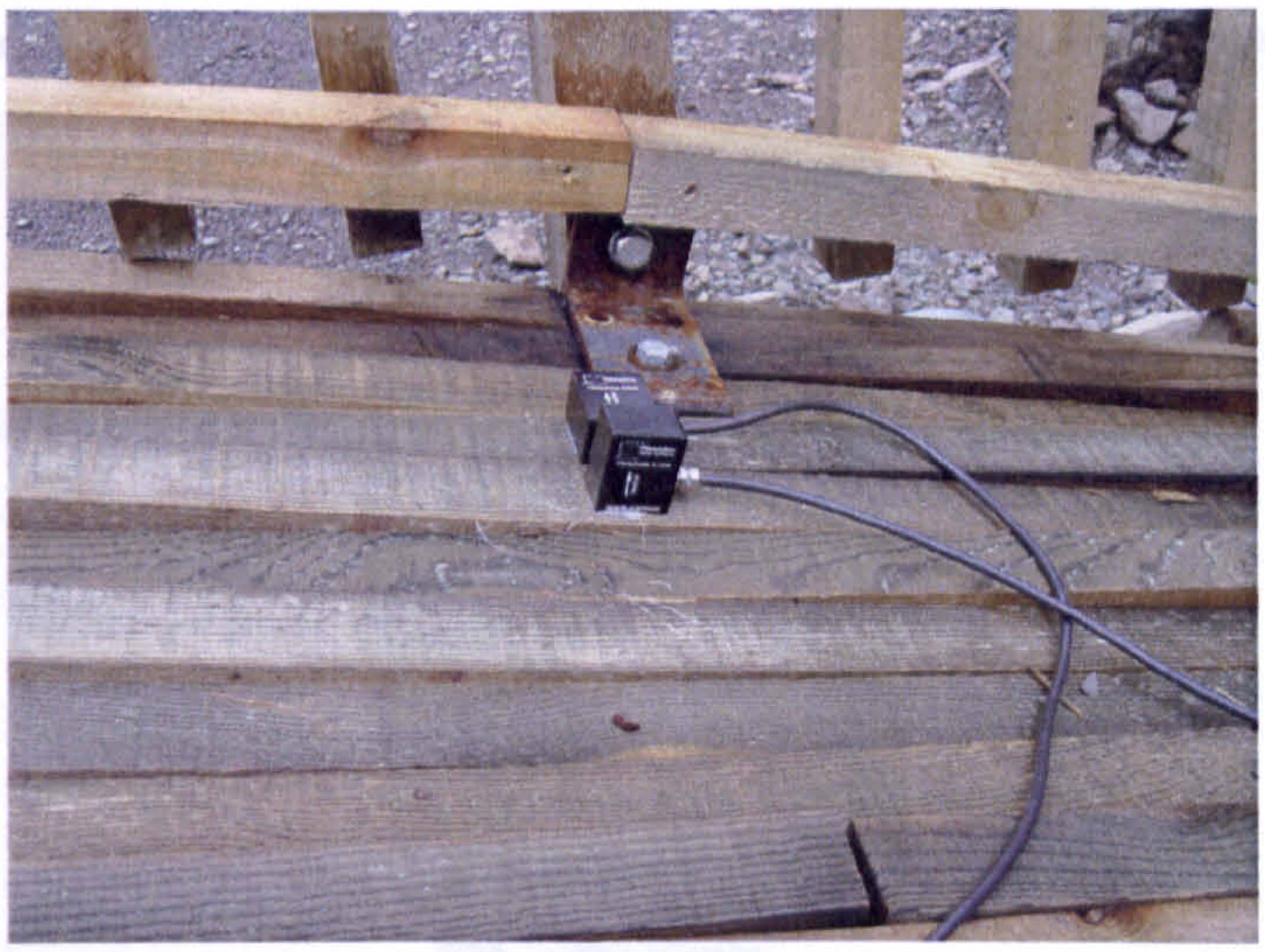


Fig 6.12 – Accelerometers for dynamic tests

The second set of dynamic tests involved the use of a dual spectrum analyser and impact excitation method. The impact hammer was used to excite the structure. The response was recorded using two accelerometers. Both the excitation force and the response signals were recorded using a multi-channel spectrum analyser. The results were analysed both in time and frequency domains, Figure 6.13.

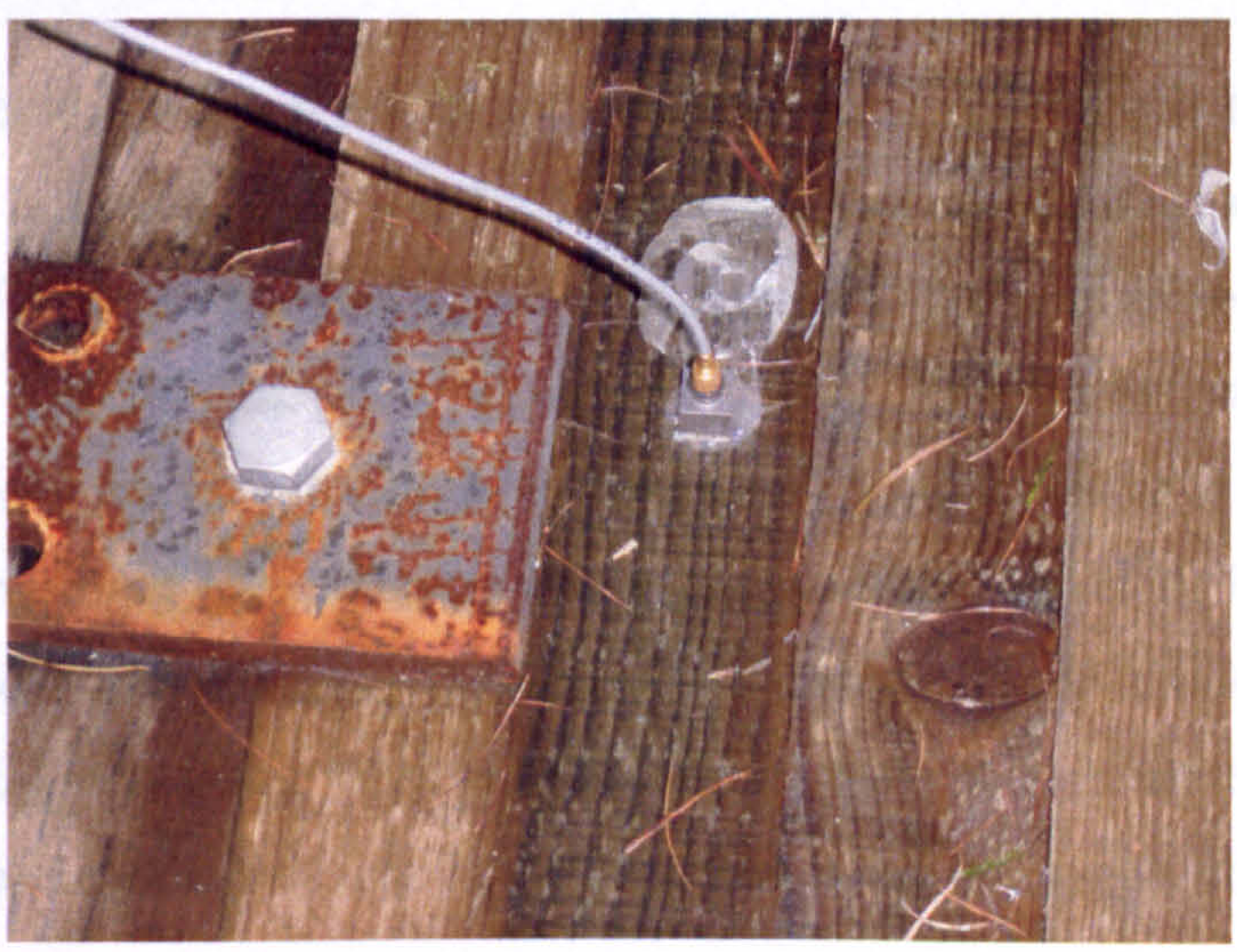


Fig 6.13 – Accelerometer – second test

The third set of tests was carried out using a versatile handheld vibration analyser RT440, developed by 'Reactec Ltd', Figure 6.14. The bridge was excited by impact (including a broadband frequency spectrum, thus exciting any system natural frequency) using 50 kg bags of ballast and the response was measured utilising the RT440 'bump test' module.

Fig 6.14 – RT 440 Portable handheld vibration analyser

This portable equipment was purchased so that the FNF of future bridges could be measured easily and quickly without the need for a separate computing facility or a mains power source. Measurements of this kind will eventually lead to load assessments which can be calculated from two sets of FNF readings with the dead load altered.

Dynamic tests could have progressed and explored accelerations and damping, but the small spans and relatively light weight decks mean that natural frequency is the only design criteria of interest. This is because it would be uneconomical to add damping to these low cost bridges. Large acceleration values, outwith the comfort zone of pedestrians, are only set up with crowd loading, which is not the norm in rural situations. When the waterproofing layer of dense bitumen macadam is added, Figure 6.15, it is bonded to the deck thereby adding to the stiffness and the NF. If it was laid on without bonding, it would just increase mass and thus reduce the NF. This can be very serious if the NF has a value near the frequency at which vandals can jump synchronously.

$NF = c\sqrt{k/m}$ Where c is a constant, k is stiffness and m is the mass.

A number of commercial bridges built through this programme had the NF taken with the portable RT440 both before and after the topping was applied. These results are shown in Chapter 7, Section 7.7.

Fig 6.15- Application of Bitmac to bridge deck

6.3 Material Properties and Component Preparation - Timber

Details of timber grading and specification are given in Chapter 5. All of the test bridges conformed to the specification in Appendices 1 and 2. The species chosen for the test bridges was Sitka Spruce because, as stated before in this document, that is the plentiful home-grown product, which this research is endeavouring to promote. The 15m span permanent bridge was built using higher grade timber.

The timber must be cut and drilled accurately to ensure that the arch fits together and the laminates settle into bearing. This has been achieved by calculating the dimensions exactly, using the spreadsheet shown in Appendix 10, and designing the bridge so that every laminate is identical and symmetrical. This has permitted an element of mass production, both in the laboratory and in the field. If the holes for the stressing bars are at 'S' mm centres then the distance from the end of the laminate to the outside holes is 'S/2'mm. The centre holes are a few millimetres above the centre line and the outside ones are a few below, which creates the arch radius. The accuracy is provided by working on a drill table with one drill position, one back stop and one end stop and spacers of thickness 6mm or so, which is the vertical distance between the holes. A typical laminate is detailed in Figure 5.3.

Care was taken to ensure that the modular size of the timbers were within a 5% tolerance. This was important for width because rogue laminates inhibit other laminates from bonding with their neighbour and thereby create a lateral discontinuity in the isotropic plate. In the USA it is common to plane one face in order to ensure accuracy of width. If the depth is out of tolerance, it affects the accuracy of the hole drilling because the mass production technique requires a laminate to be turned, which means that measurements are taken from both faces.

6.4 Materials Properties – Tension bars

The Laboratory tests used threaded mild steel bar with nuts and washers at both ends to tension the scaled-down bridges. This was very convenient because part of the testing involved the application of many different tensions and this permitted easy adjustment. However, after a number of uses at the high tensions, the threads began to fatigue. It was very difficult to attain a high enough tension on the bars in the flat decks to obtain meaningful results.

Torque was applied using a wrench, calibrated in the laboratory, to provide a specific tension in the threaded bar. This was done using the simple equipment shown in Figure 6.16. Various torques were applied and a load cell connected in series with the bar section, held in the rigid frame, provided a corresponding tension. The results were plotted as shown in Figure 6.17 so that a specific torque could be obtained for the required tension in the stressing bar. In commercial bridges, hardwood laminates are used on the outside to take the high bearing loads from the tension bars. In the laboratory, this was not done and some bearing plates, unfortunately, crushed the timbers.

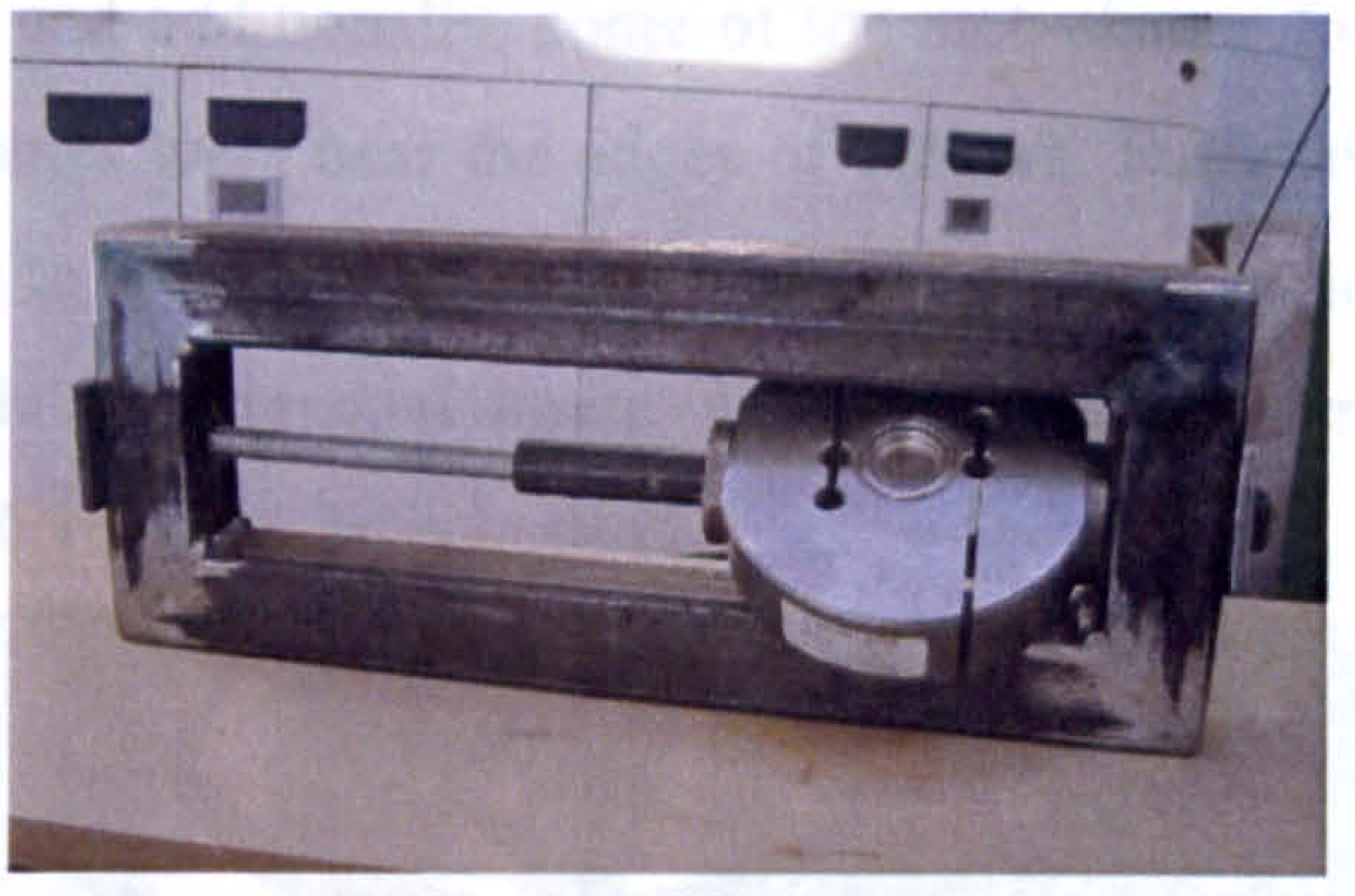


Fig 6.16 - Load cell and calibration frame

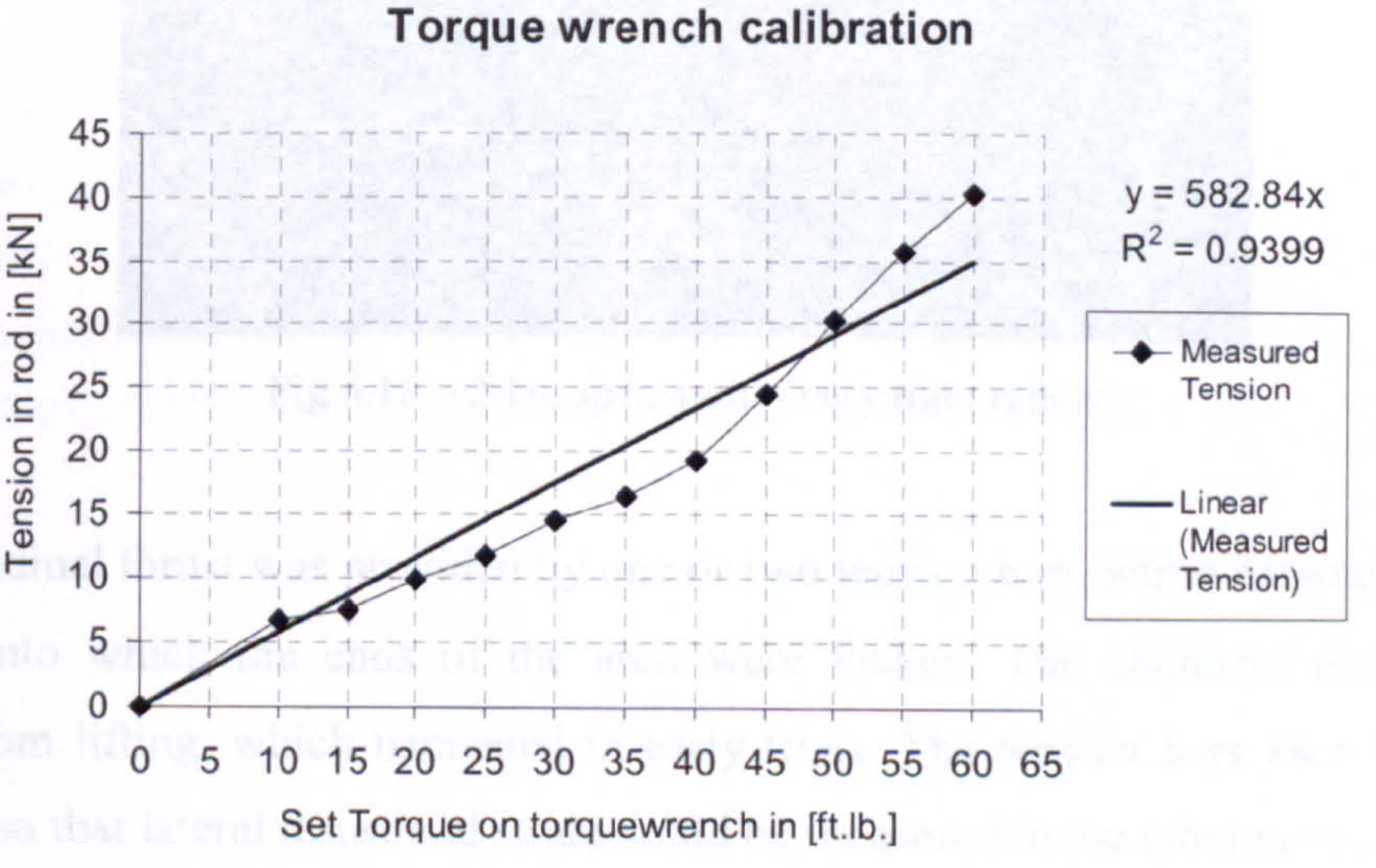


Fig 6.17 - Torque to tension plot

The full scale bridges were tensioned using either prestressing bar or Gewi galvanised HYS bar. The procedures are detailed in Section 5.3.9 of Chapter 5.

6.5 Construction Details of Structures and Restraints

A number of practical aspects had to be considered before completing the design of the test bridges. The laminates were scaled down resulting in the rod size being proportionately large and additionally, some of the arch shapes formed very tight radii. This meant that the holes were near the edges of timbers. Eurocode 5 [51] requires that prestressing is central to a laminate, but this is not possible in arches. To ensure that this was within reasonable experimental limits, shorter laminates were used for the small radius arches, Figure 6.18.

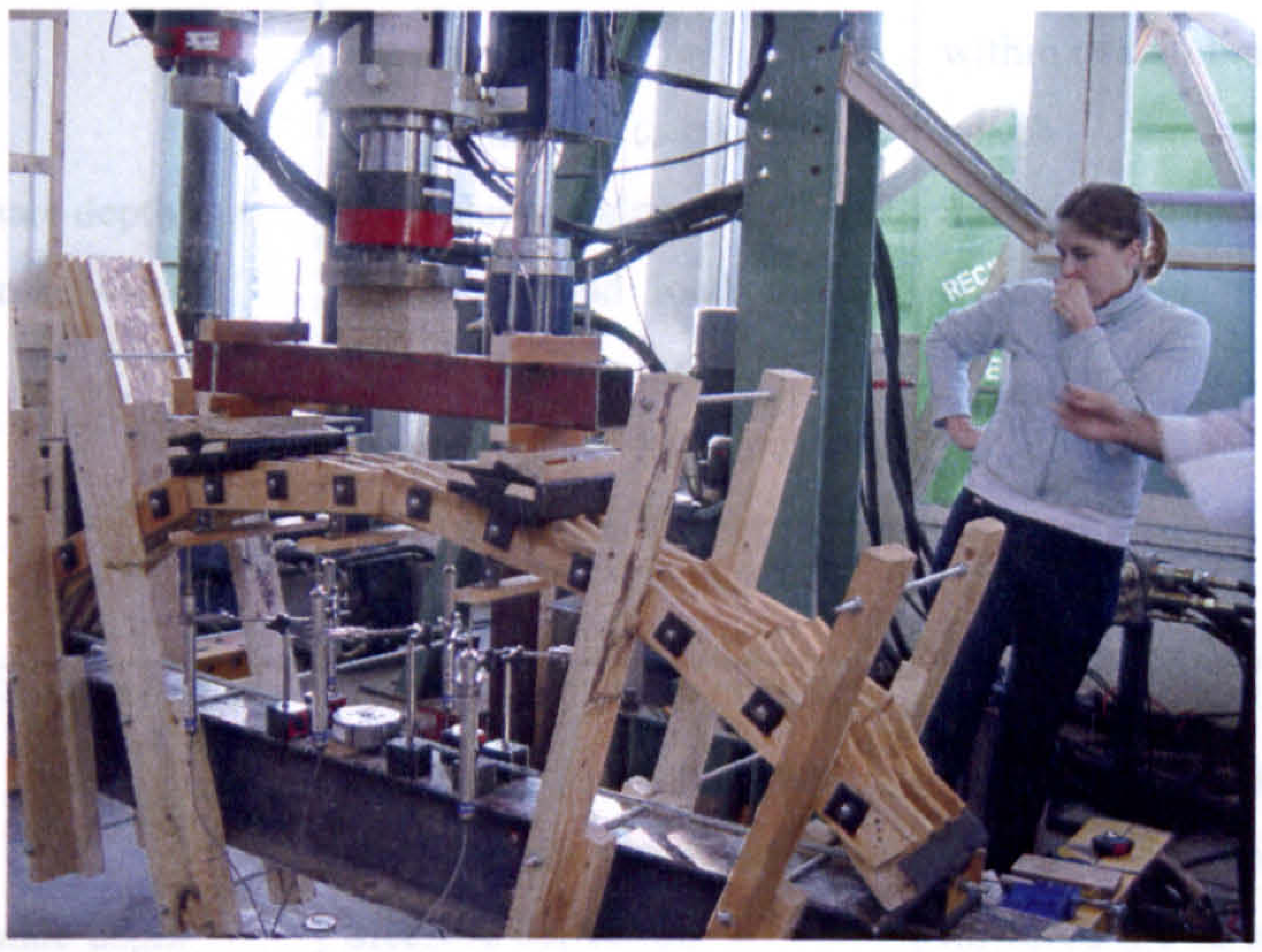


Fig 6.18 – 2.1m span arch – very tight radius

The longitudinal thrust was provided by one or two tension bars acting between two steel channels, into which the ends of the arch were locked. The channels restrained the bearings from lifting, which happened in early trials. The tension bars each had a load cell in line so that lateral thrust and strain could be measured in the laboratory, Figure 6.9. However, the thrust was too large for instrumentation in the full scale bridges so strain gauges were used.

In all tests pre-loading was essential to allow the structures to settle into a load-bearing position. An early test with the 15m span bridge, built and tested in the University car park, showed that considerable error could be introduced if this was not done.

6.6 Summary of Arch Testing

In the following sections of this chapter the testing programme, with many of the results, is presented in chronological order. Each stage of the research was prompted by findings from the previous stage. In order to achieve a logical layout, Table 6.1 shows a summary of the bridges tested and the important findings.

Table 6.1 – Summary of the tested bridges

Test	Bridge Details	Test Details	Behaviour	Failure
1	6m span Trial Arch 0.5m rise 1m wide 0.1m laminate depth 0.05m laminate width	4 point, ¼ point and line loading at 0.75m centres. Horizontal ties to take thrust. Transducers along span for deflection and load cell in ties to measure thrust. Single tension in lateral ties to give $1N/mm^2$ between the laminates	Elastic behaviour within design limits which was mirrored the results from QSE	Failed when one laminate split giving a FoS of >4 when compared to design load
2	15m span Trial/Permanent 1.25m rise 2m wide 0.25m laminate depth 0.05m laminate width	10kN sand bags to give central and ¼ point loading. Transducers along span to measure deflection. Single tension in lateral ties to give $1N/mm^2$ between the laminates	Elastic behaviour within design limits which was mirrored the results from QSE	Not loaded to failure
     3 4 5 6	2.1m span – 4 rises 0.216m wide 0.07m laminate depth 0.018m laminate width 4 rises Flat bridge rise = 0 Rise = 0.335m Rise = 0.445m Rise = 0.580m	4 point loading and ¼ point loading. Horizontal ties to take thrust. Transducers along span for deflection and load cell in ties to measure thrust. All bridges were tested with a minimum of 4 different tensions. Results showed that ⅓ of design tension was sufficient for lateral load transfer	Elastic behaviour within design limits which was mirrored the results from QSE when tension was sufficient.	Failed with FoS of 4

7 8 9 10 11	6m span – 4 rises 0.475m wide 0.95m laminate depth 0.025m laminate width Flat bridge 1 rise = 0 Flat bridge 2 rise = 0 0.25m 0.5m 1.0m	4 point and ¼ point loading. Horizontal ties to take thrust. Transducers along span for deflection and load cell in ties to measure thrust. All four bridges were tested with 4 different tensions. Results showed that 1/3 of design tension was sufficient for lateral load transfer	Elastic behaviour within design limits which was mirrored the results from QSE when tension was above 7kN	Not loaded to failure yet
12	20m span 1.67m rise 2.0-3.0m wide (varies) 0.2m laminate depth 0.05m laminate width	9kN sand bags to give central and ¼ point loading. Transducers along span to measure deflection. Single tension in lateral ties to give $1N/mm^2$ pressure between the laminates. Tested at 4 month intervals with different tensions	Elastic behaviour within design limits which was mirrored the results from QSE. Improved stiffness shown with increase tension	Not loaded to failure

6.7 6m Trial Arch and 15m Trial/Permanent

A 12m span stress-laminated timber arch bridge was designed and submitted to a design competition early in 2002. The remit was for an innovative timber bridge for all ability users and horses. The site was part of one of the government's Capital Modernisation Fund (CMF) projects, near London. These were Central Government funded initiatives, aiming to increase access to the countryside in areas of large population. The design won the competition and was immediately scheduled for construction but, unfortunately, the project ran into difficulties with land ownership and delays resulted. As the construction of the bridge had been delayed, it was decided to build a test arch for demonstration purposes. The aims were to explore the construction techniques and to determine the structural behaviour and performance of this new design (stress-laminated timber arch bridge), as it would be the first bridge of this kind to be constructed. The demonstration bridge was built as a half scale model, Figure 6.19, with a 6m span, 1m wide and 0.5m rise, using 50mm×100mm×1000mm laminate timbers in place of the full scale bridge. The full scale bridge would have a 12m span, 2m width and 1m rise with 50mm×200mm×2000mm laminates.

Fig 6.19 – First SLT arch

Whilst awaiting the testing of this half scale bridge, an opportunity arose to build a 15m span bridge on another CMF site, near Manchester. This bridge was designed, using elastic analysis and a generous factor of safety, as an arch with a 1.2m rise, 2m width and timber laminates of 50mm×250mm×2150mm. It was decided to build this at the University, under controlled conditions, then load test it before transporting it to the site in Manchester.

6.7.1 Design Details

The bridges were designed for a uniformly distributed load of 3.2kN/m^2 using grade C24, FSC certified, timber even though the ultimate goal is to use C16 or C18, as these are the grades readily available from home grown produce.

Grade D40 hardwood was used for the outer leaves of the 15m span bridge to resist the very high bearing stresses from the tensioning bars. The timber for the 6m demonstration arch was entirely softwood and consequently the outer leaves were subjected to considerable bearing deformations.

6.7.2 Preparation of the Timber

For the arch construction, a single size of timber was used and staggered in groups of four to form a module which would repeat itself throughout the deck. Four holes were drilled in each timber at an appropriate diameter which would easily accommodate the bars, and for the 15m span, their sleeves. The holes were drilled on the radius of the circular curve of which the deck was a segment. The timbers were then pressure treated with a Copper Chromium and Arsenic (CCA) compound, to repel fungal attack.

The timbers laminate lengths were chosen to allow a curve, without too much of their ends protruding above the deck surface or the holes being too far off the centre of the laminate.

6.7.3 Tension Bars

The generic aspects regarding specification of tension bars are described in Sections 5.3.2 to 5.3.9 of Chapter 5. The important general factor is that timber shrinks seasonally and the stressing bars must maintain a minimum tension so that sufficient friction is set up between neighbouring timbers, so ensuring that load is shared laterally by all timbers forming the deck. High tensile 'Dywidag' ribbed bars were used for stressing the timber sections together. They were tensioned using a hydraulic jack and locked with specialist nuts at either ends. The bar diameter was chosen so that 70% of yield stress gave the required design load to set up the friction. The bars were 15mm in diameter for both 6m and 15m span bridges. To protect the bars in the 15m span permanent bridge they were sherardised and then greased and sleeved to the same specification as ground anchors. For later bridges, 90% of yield was chosen to maximise strain. Figure 6.20 show a bundle of' Dywidag', bars sleeved ready for use.

Fig 6.20 – Stressing bars, greased and sleeved

In order to distribute the nut tension onto the timber sections 100mm square mild steel bearing plates were used for the 6m span bridge and 200mm diameter galvanised mild steel plates for the 15m span bridge.

6.7.4 Testing and Measurements

The 6m span bridge was supported on a steel frame constructed from two angle sections (one at each end), acting as bearing plates for the ends of the bridge. The horizontal thrust was contained by two steel hollow sections of 40mm × 40mm × 5mm thick (one on each side) with ends being welded to the bearing plates. The 15m span bridge was supported in a similar way but with the horizontal thrust being contained by steel tie bars attached to steel channel restraints at the ends of the arch. These were designed to eliminate the need to form rigid supports - and to provide a means of measuring the thrust by using a pair of load cells in the case of 6m span bridge, and strain gauges in the case of 15m bridge.

Deflections were measured at various points along the arch by means of displacement transducers. All readings were automatically recorded using a data logging system.

6.7.5 6m Span Arch Bridge

The 6m span bridge was tested in the laboratory using a Dartec Modular 9500 which is a combined loading and data acquisition system with a 300kN load capacity. First the bridge was subjected to a four-point loading condition as illustrated in Figure 6.21.

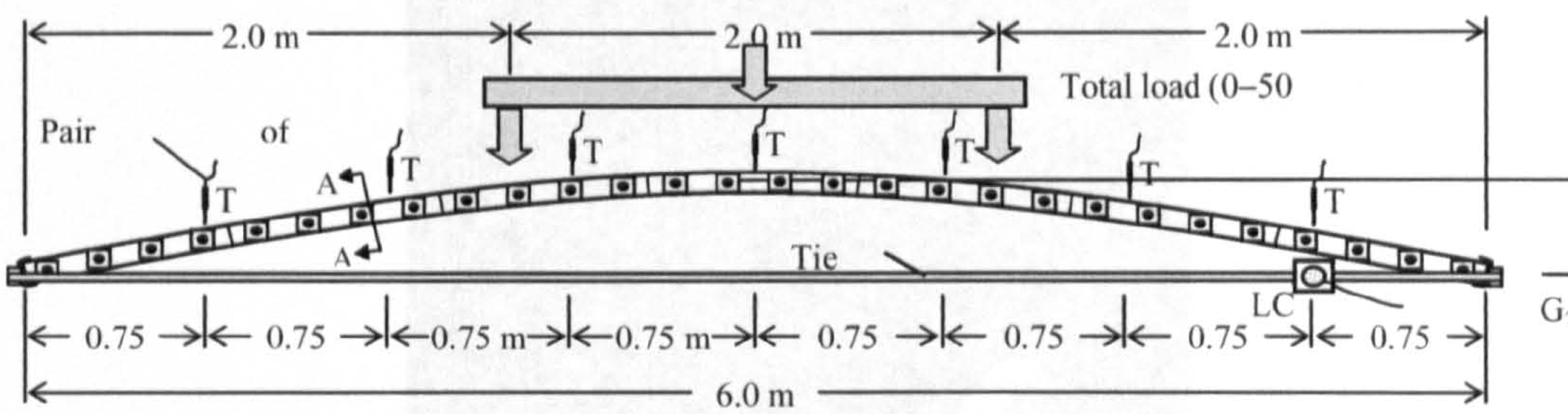


Fig 6.21 - 6m Span trial arch test set up

The bridge under preparation for testing is shown in Figure 6.22. A preload of 5kN was applied and removed a few times to eliminate any initial settlements. The bridge was then loaded at a constant rate to 35kN, released and then loaded again to a maximum of 50kN after which, the load was removed and the bridge was allowed to recover. In Figure 6.23, the load deflection / relaxation of the bridge at various positions along the length of the arch are shown and the exaggerated deflected profiles of the bridge at various load levels are shown in Figure 6.24. The second part of each position graph shows the arch recovery deflection. It can be seen that the initial test showed that much of the deflection took time to recover whereas in subsequent loadings the recovery was faster. This was because the 5kN pre-load was not sufficient to complete the initial settlement.

The support frame used for this bridge for testing purposes, proved to be very effective in preventing its ends from spreading apart, hence preventing undue increase in vertical deformation and reduction in horizontal settlement. The bridge sustained approximately 2.6 times its design load (3.2kN/m^2 on 6m^2 = 19.2kN), applied as two line loads (total 50kN) at its middle third points, without signs of any distress. At the design load, the maximum deflection of the bridge at its mid span was less than 0.003 times its length.

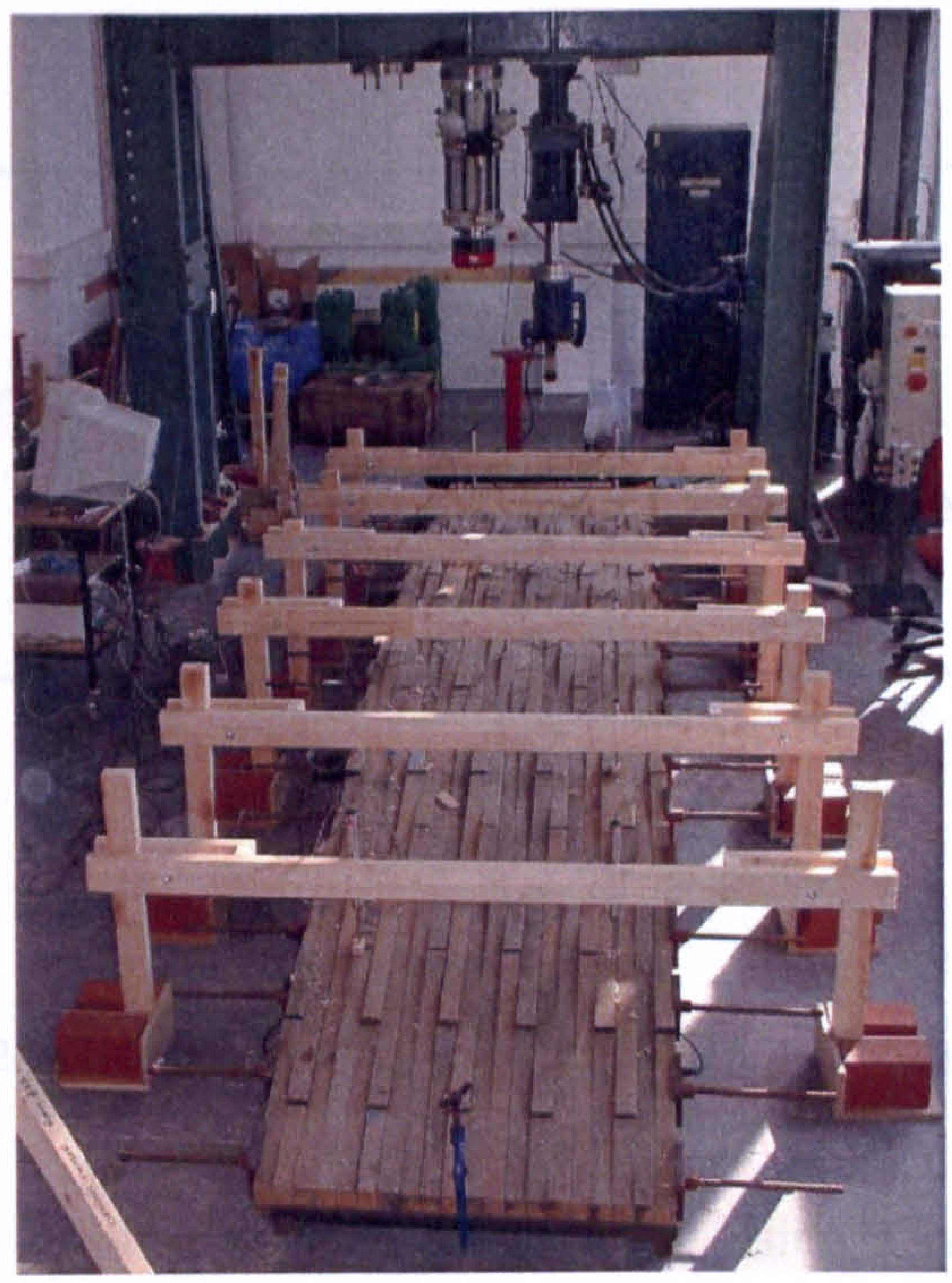


Fig 6.22 - 6m span ready for testing

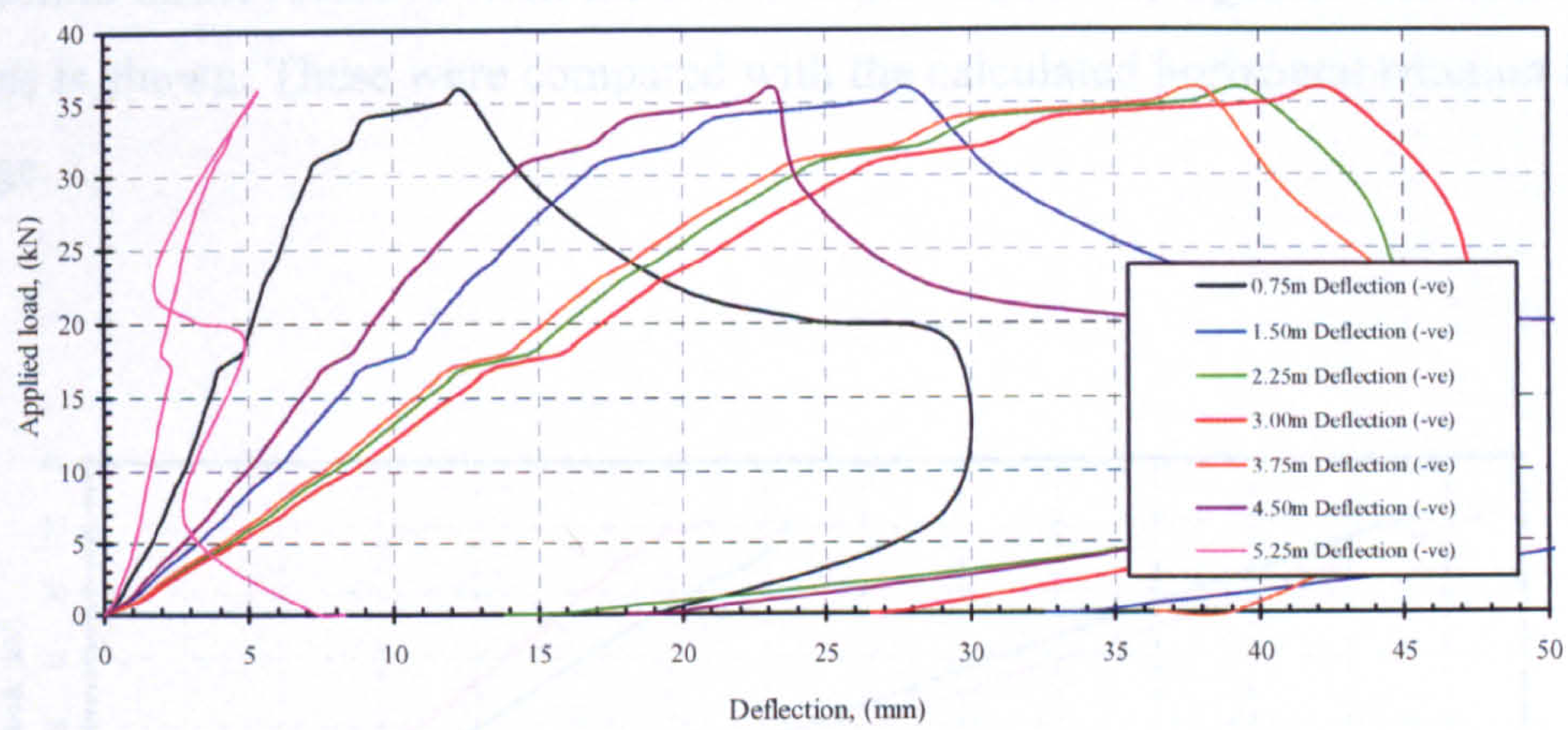


Fig 6.23- Vertical deflections at regular sections – 4 point loading

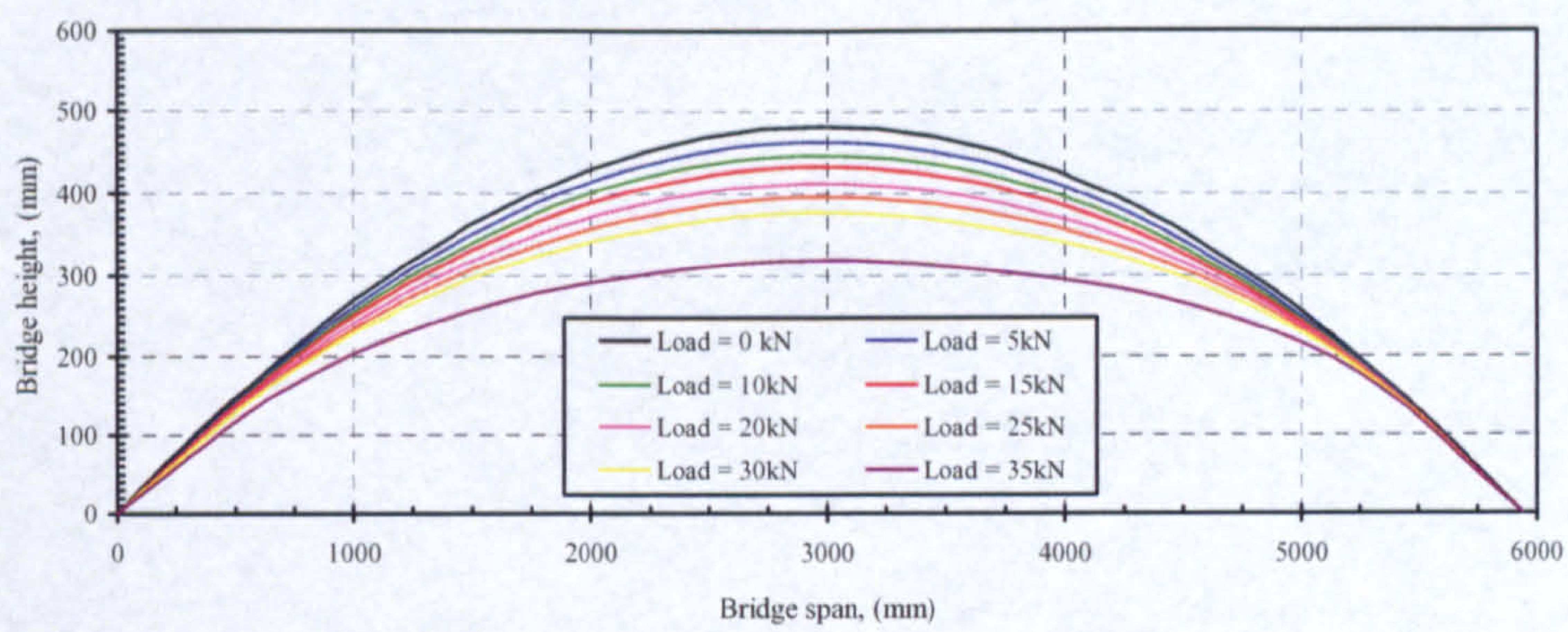


Fig 6.24 - Exaggerated deflected profile – 4 point loading

Comparison of the test results with the calculations, using a series of simple static analyses, indicated that the bridge behaved elastically up to well above its design load and sustained its arching action during the increased loading. In Figure 6.25, the horizontal thrust recorded from the two attached load cells, together with their combined values is shown. These were compared with the calculated horizontal reaction in the arch bridge.

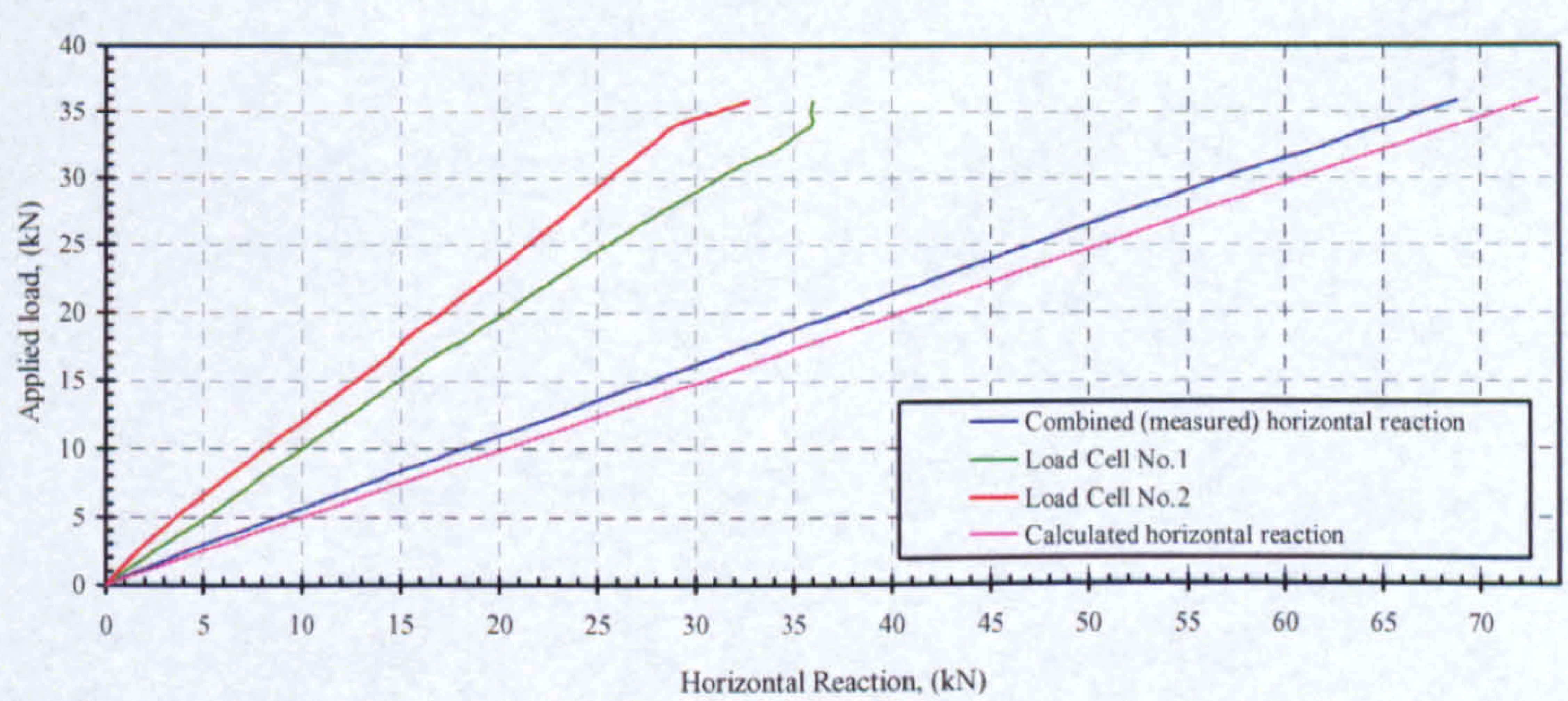


Fig 6.25 - Horizontal thrust

In the second series of tests, one support was permitted to move horizontally by 25mm, using a specially devised adjusting mechanism attached to the tie bars, to simulate settlement of an abutment and the arch was again loaded under a four-point loading condition. The horizontal movement (settlement) of the support illustrated its profound effect on increasing the vertical deformation of the bridge. In Figure 6.26, the load deflection /relaxation of the bridge at various positions along the length of the arch is shown.

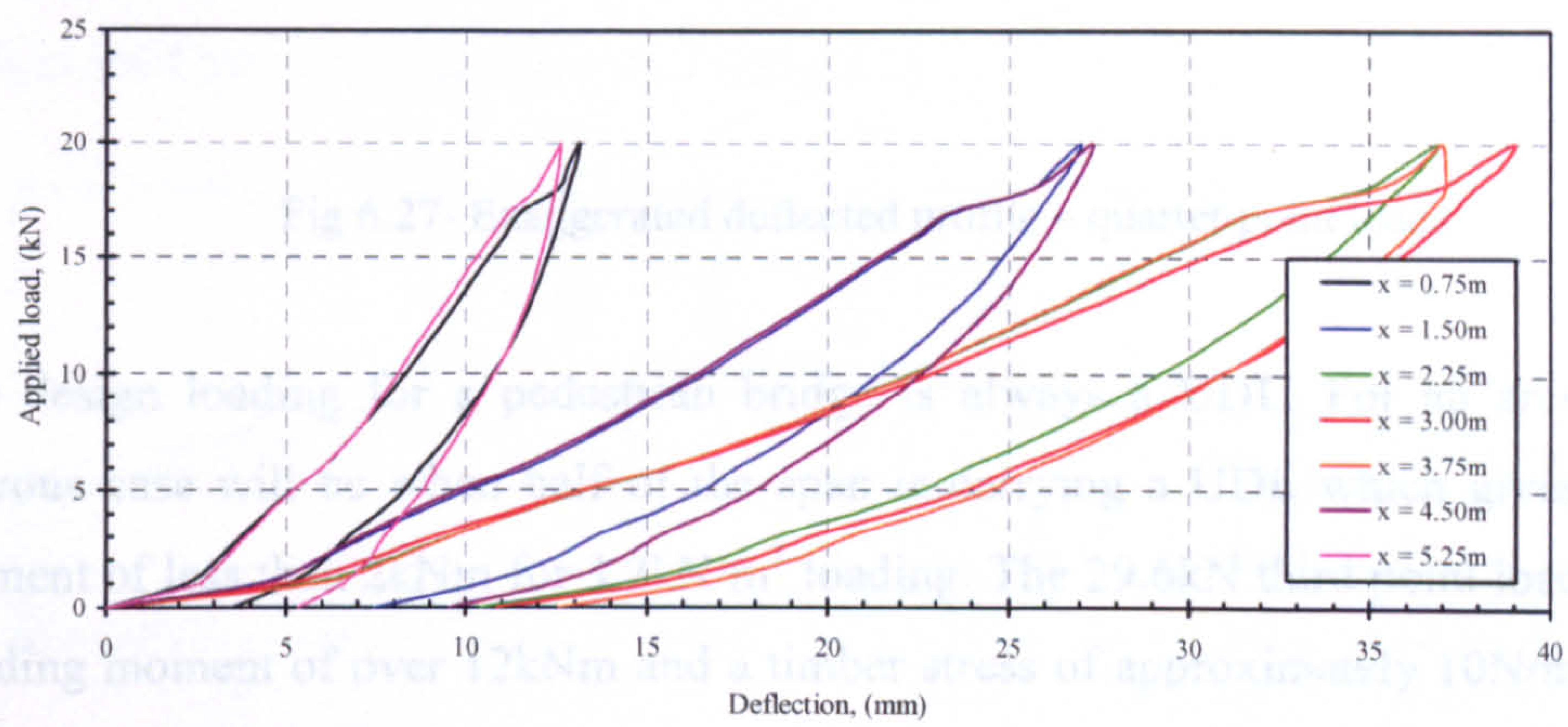


Fig 6.26 - Vertical deflections - 4 point loading and support settlement

In the third series of tests, point loads (line loads) were applied at 0.75m from a support, then at 1.5m, 2.25 and finally at 3.0m positions. In each loading position, loads were increased up to a maximum of 15kN and the deflections were recorded.

In the final series of tests, the bridge was subjected to a point (line) loading at 2m from a support and was increased until failure occurred at 29.6kN, due to upwards bulging of the arch on the unloaded side. The magnitude of the failure load clearly indicated the considerable reserve of strength provided by the arching action in the bridge. In Figure 6.27, the exaggerated deflected profiles of the bridge at various load levels up to 15kN are shown

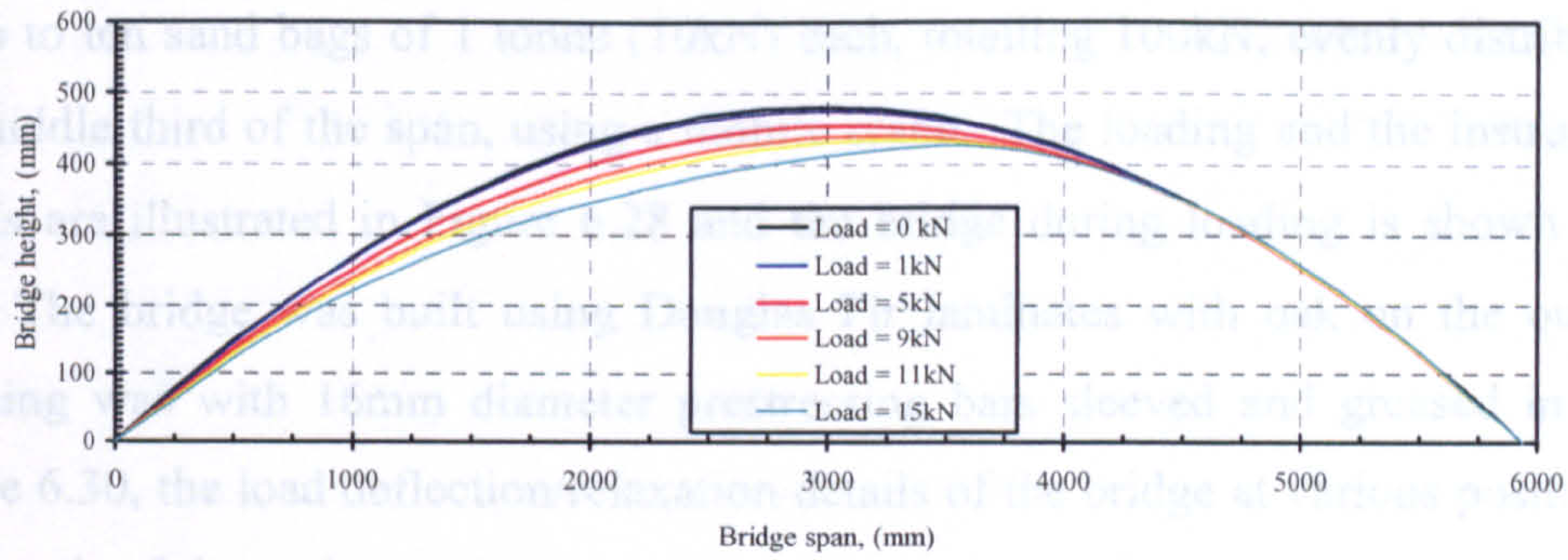


Fig 6.27- Exaggerated deflected profile – quarter point loads

The design loading for a pedestrian bridge is always a UDL. For an arch, the most onerous case will be when half of the span is carrying a UDL which gives a bending moment of less than 2kNm for $3.2kN/m^2$ loading. The 29.6kN third point loading gives a bending moment of over 12kNm and a timber stress of approximately $10N/mm^2$ parallel to the grain.

6.7.6 15m Span Arch Bridge

The 15m span bridge was constructed in a car park outside the laboratory and was loaded by up to ten sand bags of 1 tonne (10kN) each, totalling 100kN, evenly distributed over the middle third of the span, using a mobile crane. The loading and the instrumentation details are illustrated in Figure 6.28 and the bridge during loading is shown in Figure 6.29. The bridge was built using Douglas Fir laminates with oak on the outside and stressing was with 16mm diameter prestressing bars sleeved and greased in tubes. In Figure 6.30, the load deflection/relaxation details of the bridge at various positions along the length of the arch are shown and the exaggerated deflected profiles of the bridge at various load levels are illustrated in Figure 6.31.

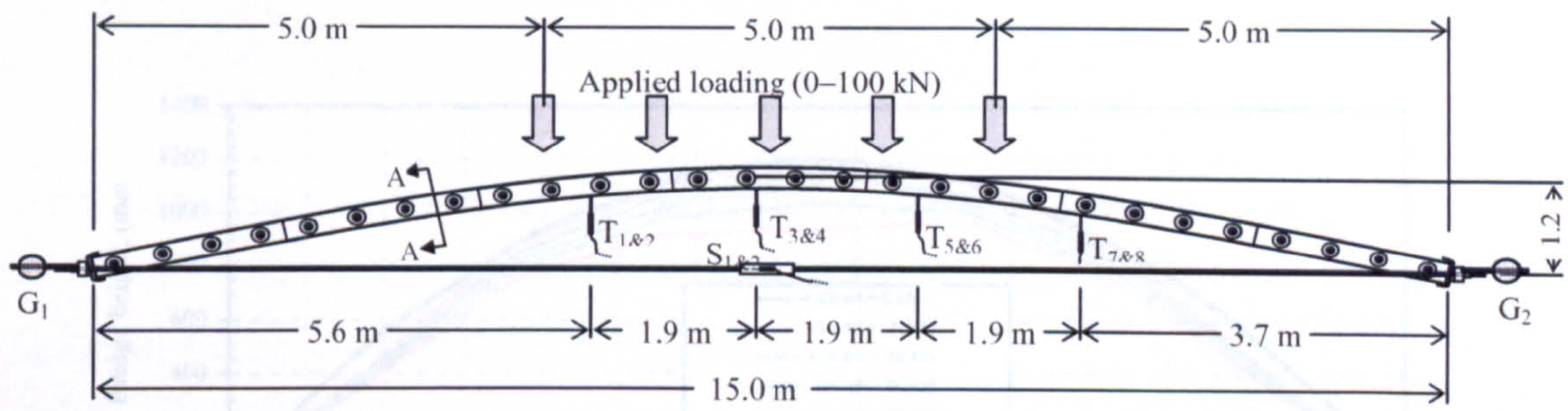


Fig 6.28 - Loading and instrumentation

Fig 6.29(a)- Applied loading 30kN Fig 6.29(b)- Applied loading 100kN

Fig 6.29 – 15m span bridge during loading

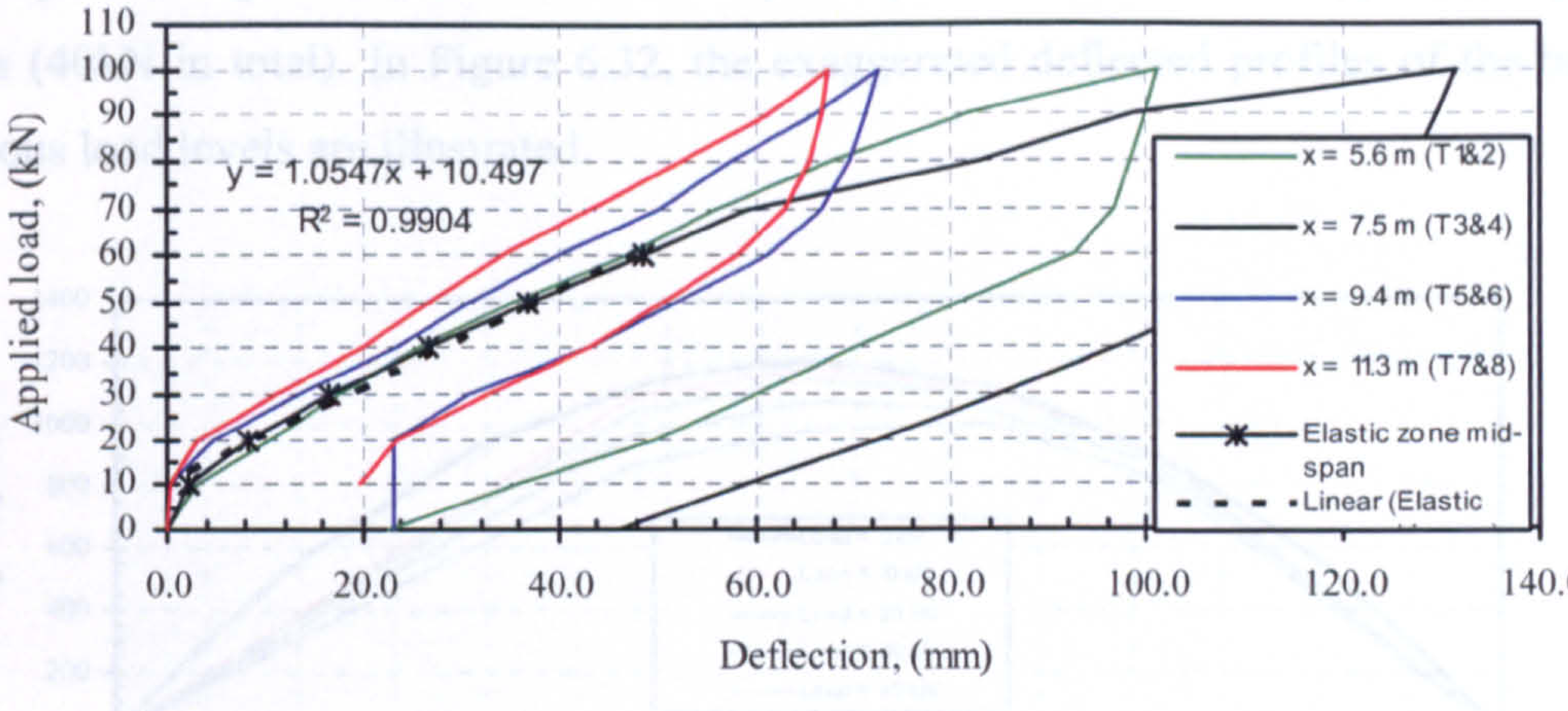


Fig 6.30 - Vertical deformation of 15m span arch

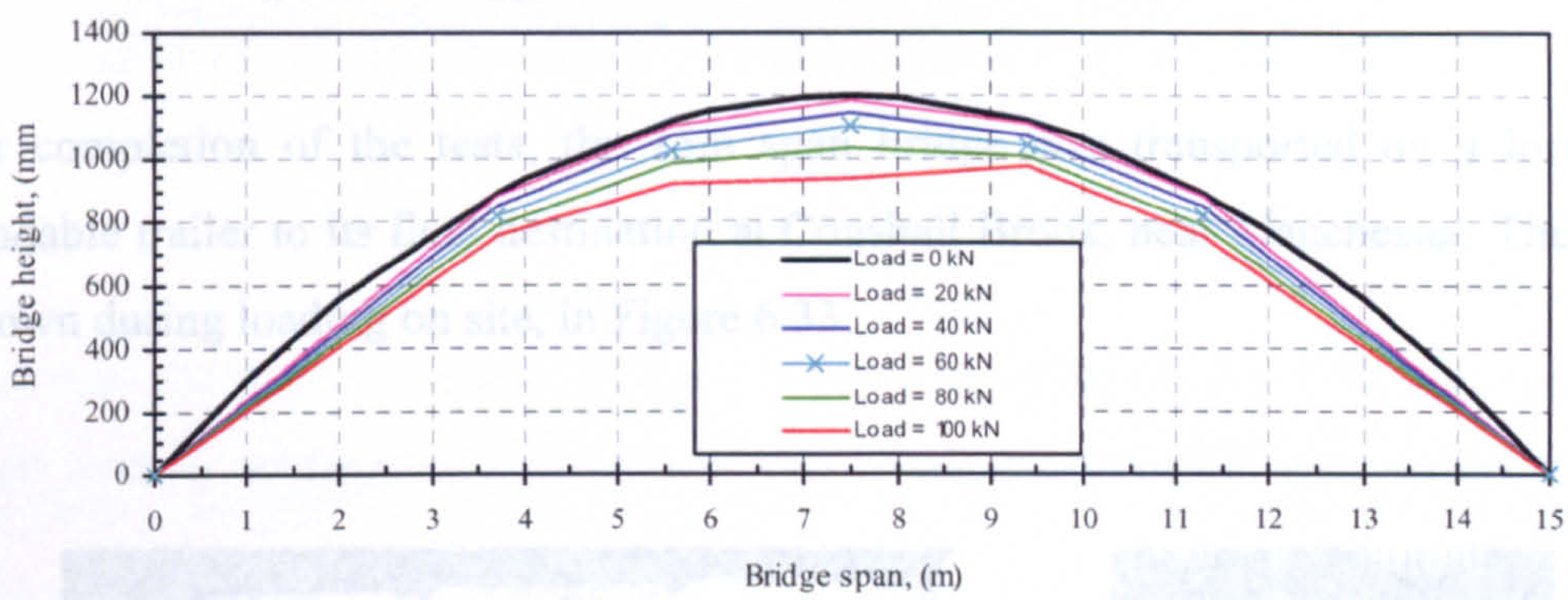


Fig 6.31 - Exaggerated deflected profile – 15m span

The support frame used for this bridge for testing purposes, was not very effective in preventing its ends from spreading apart. A total of 28.4mm horizontal movement was recorded during loading, which in turn caused undue increase in vertical deformation and also reduction in horizontal thrust. Analytical results showed that such a horizontal settlement would increase the mid span deformation of this bridge by over 13 folds. At the load of 100kN the maximum deflection of the bridge at its mid span reached only 0.008 times its span, Figure 6.31.

Finally the bridge was loaded eccentrically at a position 5m from a support, using 4 sand bags (40kN in total). In Figure 6.32, the exaggerated deflected profiles of the bridge at various load levels are illustrated.

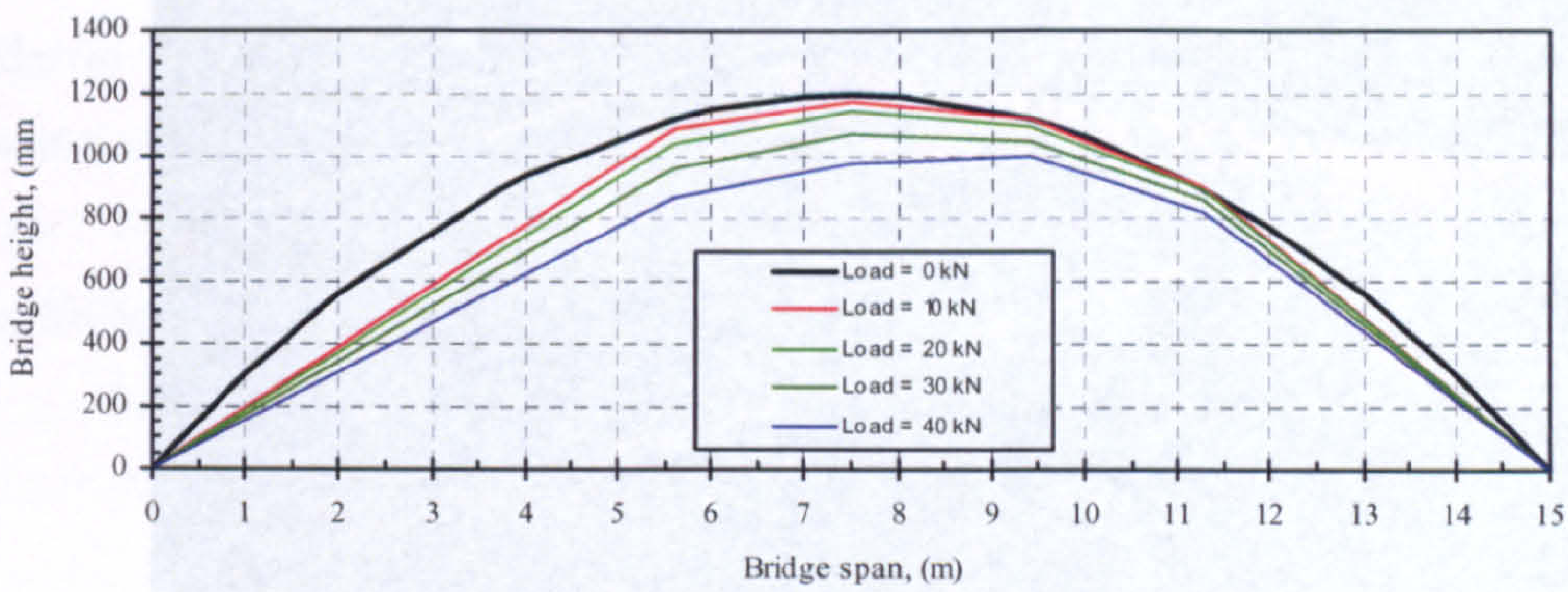


Fig 6.32 - Exaggerated deflected profile – quarter point loading

After completion of the tests, the 15m span bridge was transported on a lorry with extendable trailer to its final destination at Cogshall Brook, near Manchester. The bridge is shown during loading on site, in Figure 6.33.

Fig 6.33 – Loading 15m span onto lorry

Fig 6.34 -15m span bridge under construction on site

Figure 6.34 shows the 15m span bridge during construction on site at Northwitch. It was erected over an existing weir in a disused salt mining area which is now a country access park.

6.8 2.1m Span Arch Bridges

The tests on the 6m and 15m arches illustrated that small sections of relatively low grade softwood timber could effectively be used to design and construct large and attractive structures. The method utilises the better qualities of timber in compression and end-bearing, than its relatively poorer one in bending, in an arching action which contributes significantly to the overall strength and stiffness of the bridges.

The utilisation of an arch profile in the UK, in design of the already successful stress-laminated timber bridges, elsewhere, provides enhanced and excellent performance and endurance when constructed and maintained properly.

The ongoing research in this programme was now planned to explore the full potential of stress-laminated arched timber structures with regard to factors affecting structural performance and stability, assembly and construction processes, maintenance, durability and life-long reliability. End of life disposal/recycling issues also require consideration in order to develop a comprehensive design guidance and a database of the key design, construction and maintenance issues for forestry and construction industries.

To this end it was decided to load test a set of laboratory scaled arches of:-

2.1m span

With 4 rises = 0 flat
335mm
447mm
580mm

For 4 lateral tensions = Finger tight
3.83kN
7.66kN
11.49kN

To explore the load deflection relationships and the differences in stiffness due to different lateral tensions.

From the work so far, the most significant differences between flat decks, which have been well researched by others, and arches, were the extra stiffness from the arch rise and the influence of the lateral stress. It was considered that the compressive action of the arch rendered the lateral tension less crucial than in flat decks, which collapse if too much tension is lost.

6.8.1 Loading

Although this was not a full scale structure, a reference loading of 3.2kN/m^2 was used to quantify the capabilities of the experimental arches by applying some factor of safety. This is the loading which was used to design the other full scale bridges and is the accepted footbridge crowd load for the countryside [19].

A similar loading pattern to the trial 6m span was used. Four point loading was applied as two equal line loads at mid span 700mm apart (⅓ span). This would simulate the UDL. The loading was applied incrementally up to a load within the elastic limit. A further set of tests were carried out with a point load at the ¼ point of the span. Again an incremental load but this time it was applied to failure.

6.8.2 Preparation of Timber

The timber laminate size used for all arches was 17.5mm wide × 70mm deep × 530mm long to give a scale factor to the normal 50mm thick timber used for full scale structures. Each laminate had 3 No. 16mm diameter holes drilled on a curved centreline at the radius of the arch and about the centreline of the laminate. The timber was dried to a moisture content of approximately 15%. Each bridge comprised twelve laminates wide, each staggered in sets of three, so that at any cross section there would be four butt joints and eight solid timbers. The effective cross section was therefore ⅔ of the actual section.

6.8.3 Tension Bars

The first tests had used HYS stressing bars which were tensioned using a hydraulic jack. This was time consuming and as the tension would have to be adjusted many times, it was decided to use galvanised mild steel threaded bars with nuts and washers at each end. The smaller scale arches would not require such large lateral forces as are used for full scale bridges. This was another reason for using threaded bars. It was also considered as a possible permanent stressing system for small arches therefore the stressing system would also be under test. Figure 6.35 shows the beginning of construction of a small arch using threaded bar. The first arches were built from the abutments and from one side. This led to difficulties as the tensioning rods were difficult to support in the temporary situation. It soon became clear that construction should begin with the mid span central laminate, as shown.

Fig 6.35 – Threaded bar tensioning – first laminate

To ensure accurate tensions, the system described in Section 6.4 was used to calibrate the torque wrenches used to tension the bars. The system worked very well for the low tensions but at loads of 20kN, the washer compressed the external laminates, indicating bearing failure and the threads eventually began to fatigue. The 20kN tension produced approximately $1N/mm^2$ between the laminates.

6.8.4 Testing and Measuring and Equipment

The arches were mounted in a frame of steel channel sections, connected by a length of threaded bar acting as a tension rod, to take the horizontal thrust. The bar had a load cell in line to measure the thrust load and the longitudinal movement. Four different arches, of 2.1m span were built with rises, from the tension bar to the centreline of the top laminate, of 0mm, 335mm, 447mm and 580mm.

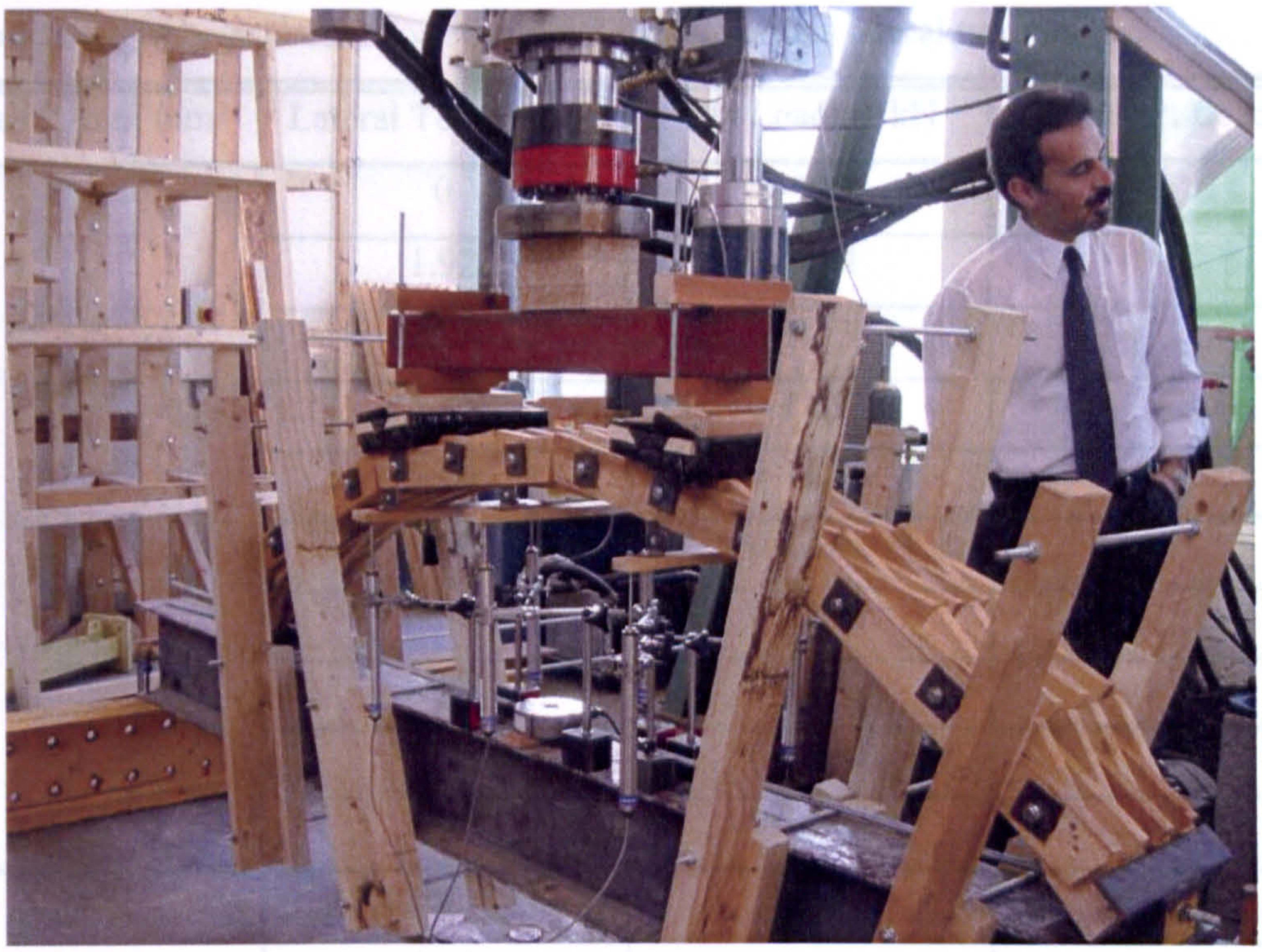


Fig 6.36 – 2.1m arch with 4 point loading

Because the arches were only 210mm wide they had to be supported laterally for safety, Figure 6.36. This would have no effect on the results. The 'Dartec Modular 9500' equipment was again used to apply the loads. The line loads of the four point loading had to act through a bed of sand to ensure the load was evenly applied. Six transducers were positioned in pairs under the mid point and the two line loads. These could not act directly on to the arch because the soffit sloped too steeply to give a reliable seating, so intermediate timber strips were added as seatings. The transducers were positioned at ¼ points and mid-span for the ¼ point loading case.

6.8.5 Tests and Results

Each arch was loaded centrally and unloaded to settle them in and bed down the structure. Each was then loaded centrally up to 10kN for four different tensions and at quarter point with maximum tension to failure. Table 6.2 gives a summary of the bridges and loadings.

Table 6.2 – 2.1m span arches - vertical loading for various lateral tensions

Rise of Arch mm	Lateral Tension kN	4 Pt Loading kN	Quarter Pt Load kN
Flat	0	0-4.38	
	1.92	0-9.44	
	3.83	0-10.54	
	5.75	0-12.92	
	7.66	0-11.86	
	9.58	0-12.33	
	11.49	0-12.68	
	13.41	0-12.47	
	15.32	0-14.81	
	17.23	0-14.45	
335	Finger tight	0-10	
	3.83	0-10	
	7.66	0-10	
	11.49	0-20	0-25
447	Finger tight	0-10	
	3.83	0-10	
	7.66	0-10	
	11.49	0-10	0-25
580	Finger tight	0-10	
	3.83	0-10	
	7.66	0-10	
	11.49	0-10	0-25

The deflections were very small, especially in the elastic zone up to 10kN loading. The deflections were of similar order of magnitude for all three arches and even a little greater for the 580mm arch. This was surprising as it was considered that it should have been the stiffest but may have had been due to poorer quality timber in this arch. However, it will be shown in Chapter 8 that there is an optimum span to rise ratio.

In each case, the deflection for finger tight deflection at full load was approximately twice the deflection for all the other lateral tensions. This showed that at minimum lateral tension, adequate friction was mobilised to permit full load transfer for all arch shapes.

The lateral settlement was not measured although the lateral thrust was. The thrust can be used to estimate the settlement as will be shown in Chapter 8.

The loads were plotted against deflections so that comparisons could be made and linear expressions could be derived for each lateral tension. The gradient would be a measure of the arch stiffness. The stiffnesses were then plotted against the lateral tensions.

Figure 6.37 shows a set of plots of load against deflection at the ⅓ span for the 335mm rise arch with four point loading. Each plot is for a different lateral thrust from the tension bars. For comparison a plot of an elastic analysis is shown for full lateral tension (dotted black line). It can be seen that this does not appear to correlate exactly with the 11.49kN lateral thrust plot which is its equivalent. No lateral settlement has been allowed for and it will be shown in Chapter 7 that the values are within the limits of experimental error.

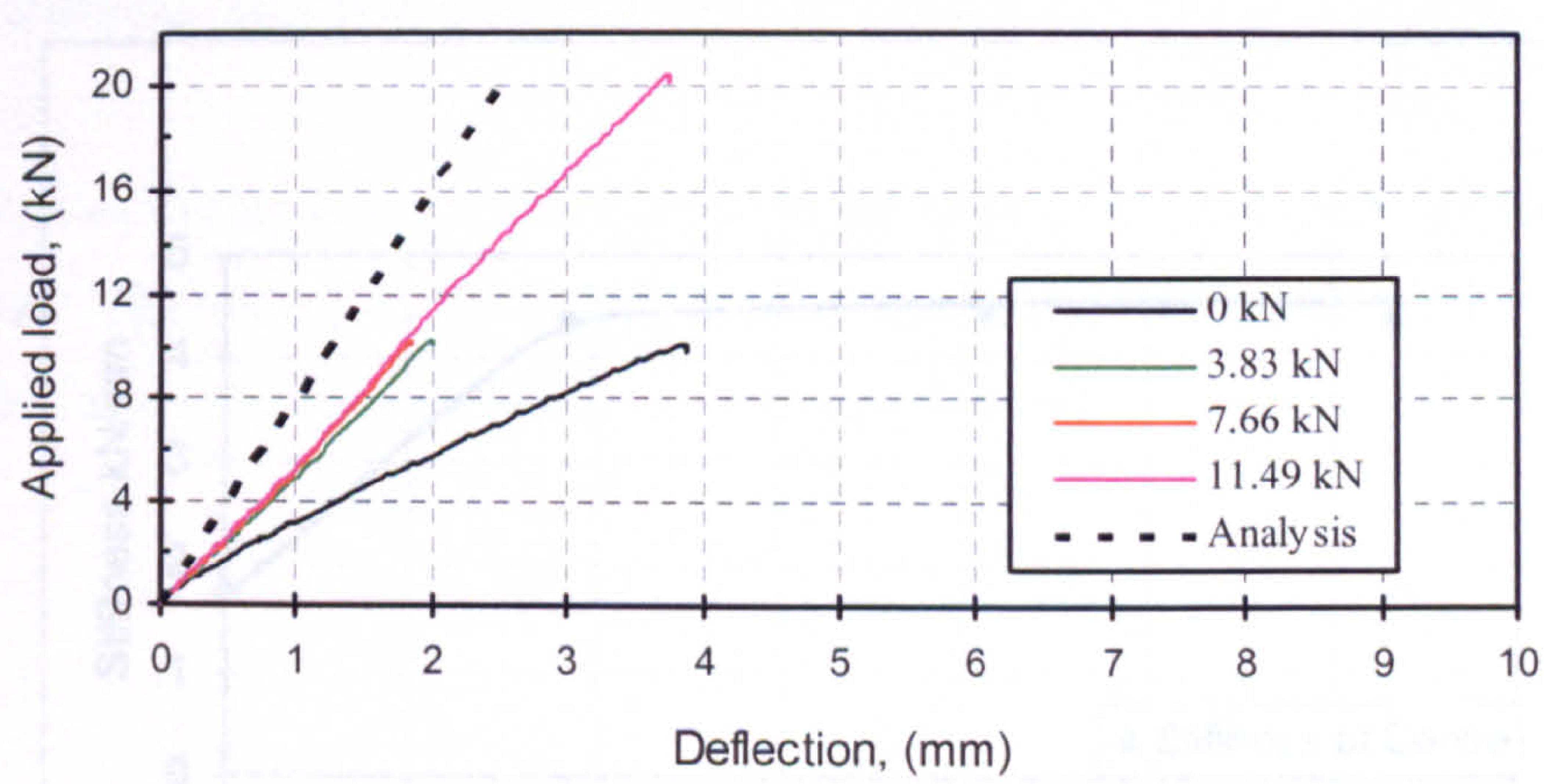


Fig 6.37 – Load deflection graph at 0.7m – 4 point loading – 335mm rise

Figure 6.38 shows a set of plots of load against deflection at mid-span for the 335mm rise arch and four point loading. A full set of similar plots for all load cases are shown in Chapter 7. Correlation is not perfect for the same reasons explained above for the ⅓ point loading.

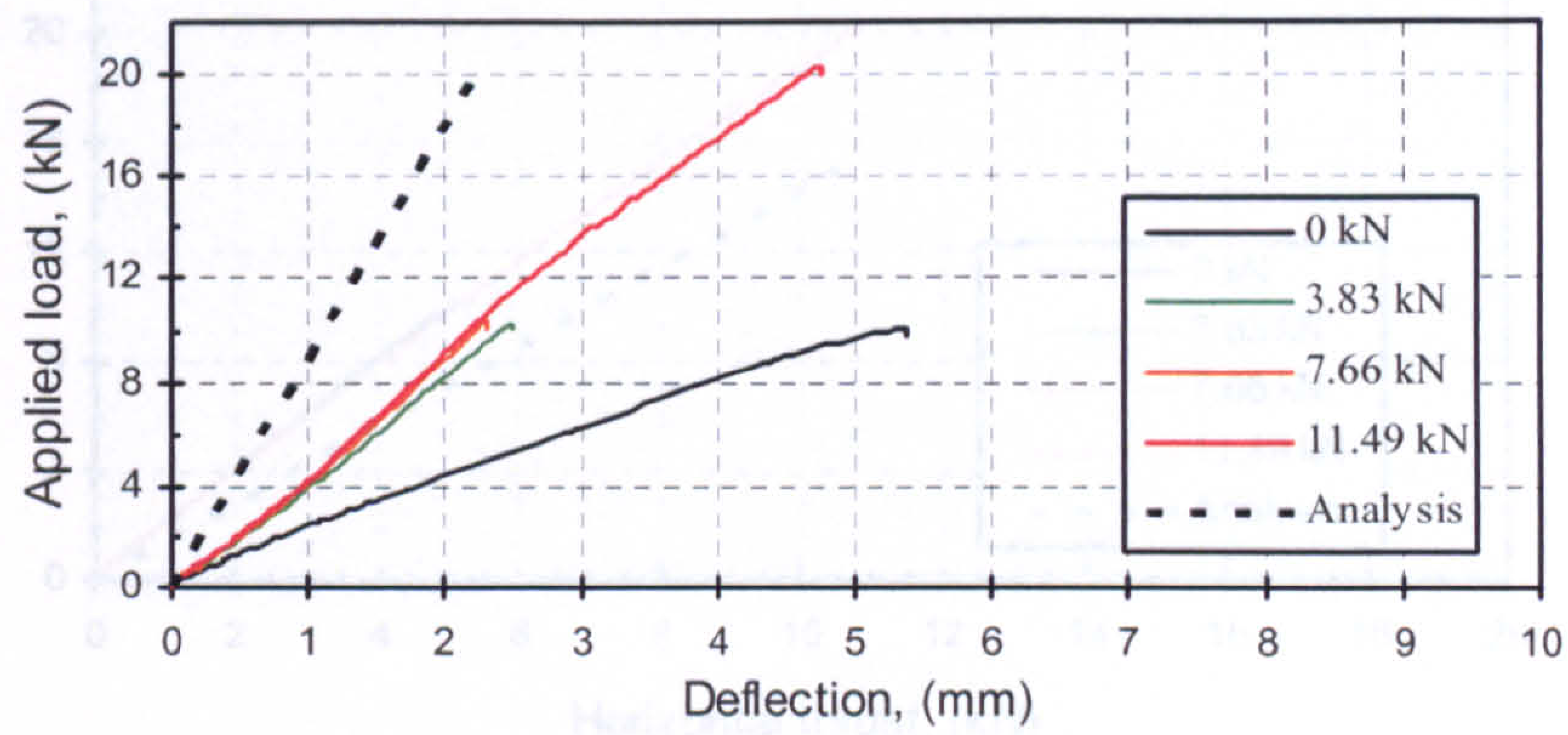


Fig 6.38 – Load deflection graph at mid point – 4 point loading – 335mm rise

Figure 6.38 illustrates the linear load deflection relationships representing the stiffness of the arches. As lateral tension increases the stiffness increases. Figure 6.39 shows these different stiffnesses plotted against their corresponding tensions for one particular arch. Again, Chapter 7 shows all of the results for the other arches.

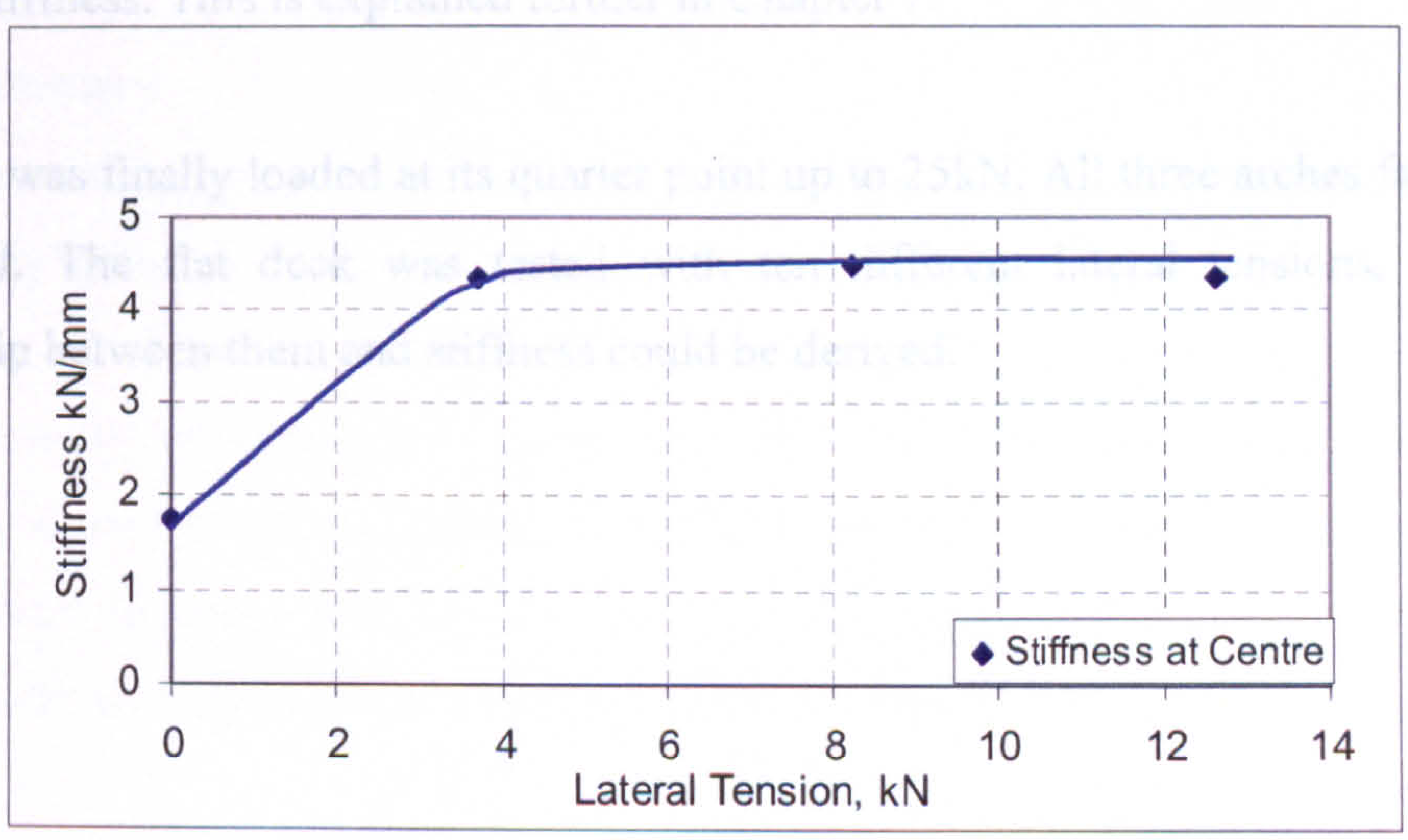


Fig 6.39 – Bridge stiffness – 335mm rise

Figure 6.40 shows a plot of horizontal thrust at the supports against applied load. The results of an analysis are also shown which correlate well with the experimental results.

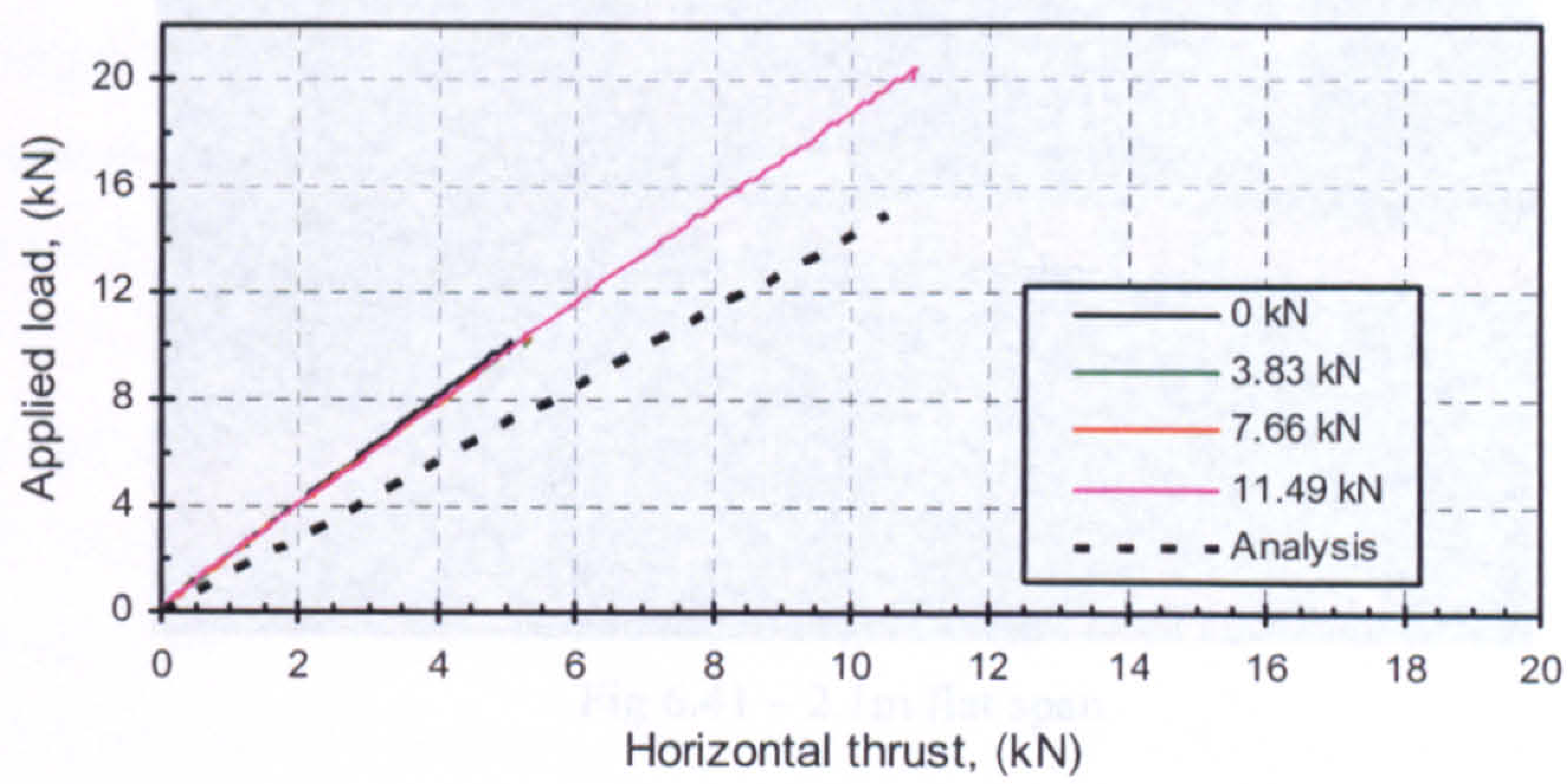


Fig 6.40 – Horizontal thrust – 335mm rise

The 447mm rise arch performed in a similar manner but it deflected a little less and the 580mm rise arch deflected more, showing that there may be an optimum for arch rise in terms of stiffness. This is explained further in Chapter 7.

Each arch was finally loaded at its quarter point up to 25kN. All three arches failed at the same load. The flat deck was tested with ten different lateral tensions, so that a relationship between them and stiffness could be derived.

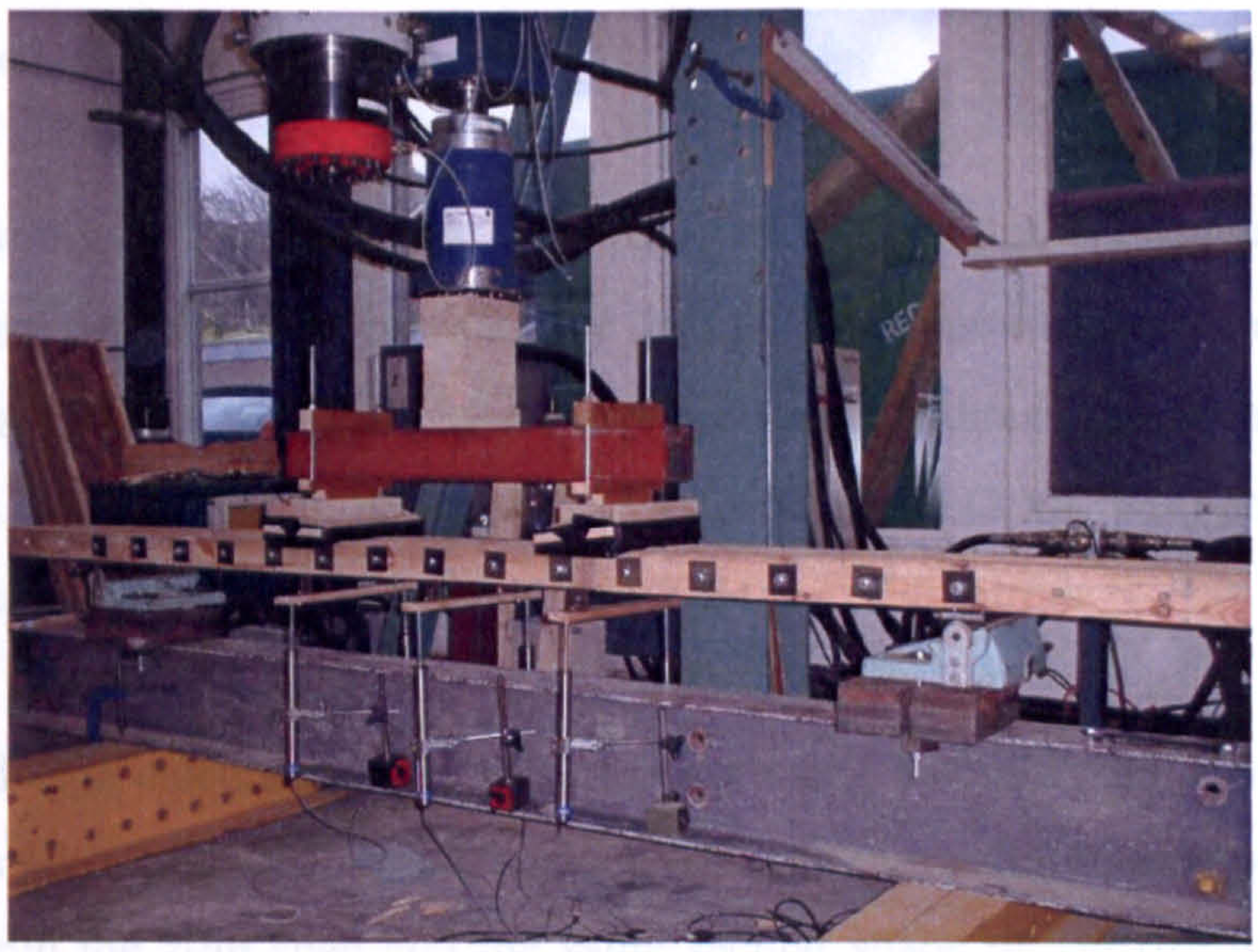


Fig 6.41 – 2.1m flat span

The tests with the flat spans, Figure 6.41, showed how un-stiff (flexible) they were at low tension and highlighted the safety which the arch bridge possesses if tension is lost in service. Later experimental work was done on the flat deck, using stressing and glue. The increase in stiffness was exponential. However, this work is not part of this research.

6.8.6 Summary

The programme of tests on the 2.1m span arches, with different rises and lateral tensions, significantly added to the understanding of the structural actions of stress laminated arches. The initial tests on the 6m and 15m span had shown that arching action took place and that results correlated with elastic analysis. The 2.1m span tests extended those findings to show that lateral tension has a significant effect on load capacity and that full strength was achieved at relatively low tension. It also showed that arch rise significantly affected stiffness and that ever increasing rise does not produce ever increasing stiffness.

These results pointed clearly to the specification for the next stage of the programme which would need to show confirmation of the findings for larger scale laboratory tests and another full scale test to confirm the results of the 15m span test.

6.9 20m Span Full Scale Test Arch

Having successfully built and tested a 15m span with 250mm timbers and established that elastic linear analyses provided comparative results, it was assumed that this would hold true for larger spans. Commercial orders were received for three 20m span arch bridges, so the decision was taken to build a test bridge to full scale (20m span) and use it as an exhibit at the June 2004 Royal Highland Show (RHS) in Edinburgh.

Analyses and design calculations were carried out using a countryside crowd loading of $3.2kN/m^2$. This permitted a 200mm deep deck (an optimum solution in regard to material availability), with the 20m span and the same secular shape of 1m rise for a 12m span giving a 1.67m rise. This geometry provided a structure with a natural frequency of just over 4.0 Hz as calculated using finite element analysis.

6.9.1 Loading

Design loading for footbridges in the UK is detailed in Section 3.5.1 in Chapter 3. The chosen loading for the design of this bridge was $3.2kN/m^2$ which is adequate for countryside crowd and horses.

However, slender structures designed for lighter loads will have low values of natural frequency which, together with low stiffness and mass, can be excited easily. In this context it is vandal loading which could become critical. Other research [70] indicates that the maximum frequency at which a small group of vandals can synchronise their jumping is 2.5Hz therefore the natural frequency of a small footbridge should be greater than this to avoid the possibility of vandals damaging the bridge by co-ordinating their load frequency with the natural frequency of the bridge, and causing resonance.

6.9.2 Stability

In order to increase the overall stability of the structure, it was decided to widen it to about 3m at the supports, reducing to a width of 2m at mid span. This would also provide a more attractive shape to the bridge. Handrails were fitted, similar to those envisaged on the final structure, as this would have an effect on the overall stiffness of the bridge.

The test bridge was first built at the RHS ground and because foundations could not be excavated, tie bars connected the springings, or end bearings, of the arch, Figure 6.42. These were formed using Rolled Steel Channels (RSC) and provided the lateral thrust. This same configuration had been used to form the lateral tie for the 15m bridge, previously tested and created some slip problems [76]. As expected, a relatively large amount of movement occurred when the tie bars holding the bridge ends together took the strain. This in turn caused a large increase in the vertical deflection of the bridge.
A great deal of care, in general, is required to ensure that the arch shape is correct from the outset. The construction at the RHS, by a team of workmen, had started off at too flat an angle to the ground, which produced a flatter profile than the design required. This resulted in significantly larger deflections and, in turn, caused a lower natural frequency. Although this was to be expected, the effect on its 'liveliness' seemed disproportionate. Here, the importance of the geometrical shape (arch profile) of the structure was highlighted, in satisfying both serviceability and strength design criteria, Figure 6.42.

Fig 6.42 – RHS bridge too flat

After the RHS, the bridge was dismantled and reconstructed by four men in five days in a quarry at Glentress, Peebles, in the Scottish Borders, about twenty five miles from Edinburgh. The foundations (mass concrete 400mm wide by 750mm deep) were anchored into the quarry base. They were deliberately not built to the size which would be used in permanent construction, in order to add a further facet to the tests. Some lateral movement was expected but it would be realistic, in terms of an actual structure and it would be measurable by establishing survey points at the springings.

6.9.3 Timber

Sitka Spruce was chosen in accordance with all the specification requirements detailed in Section 5.2.1, Chapter 5. Four holes were drilled in each timber in accordance with the details shown in Section 5.3.6 of Chapter 5. In Figure 6.43, the 20m span bridge is shown during construction. The timbers were rough sawn 50 mm thick x 200mm deep x 2m long plus 36mm to allow for curvature.

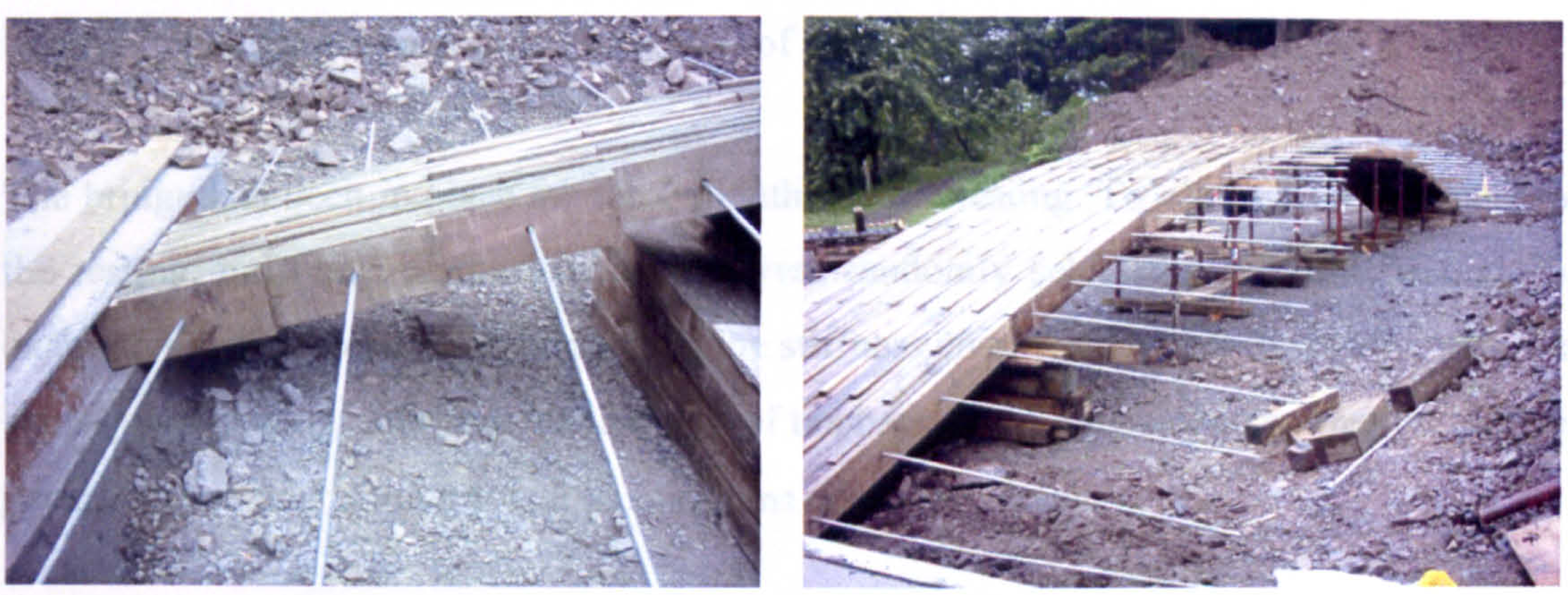


Fig 6.43 – 20m span bridge under construction

6.9.4 Lateral Stressing Bars

One of the important aspects of design for SLTA bridges is the function of the lateral stressing bars to achieve the necessary effects:

- sufficient tension to stress all of the laminates
- sufficient friction to transmit longitudinal and transverse stresses
- sufficient extension to maintain tension when laminates dry and shrink
- resistance to corrosion.

16mm 'Gewi' bars, supplied by 'Dywidag', were favoured because they are galvanised and, although their yield stress is only 500/600N/mm^2, the bar strain to maintain tension was considered less important in an arch bridge [76] than in a flat bridge. The bars were stressed to 90% proof stress providing 100kN to 200mm wide laminates at hole spacing of 500mm giving a pressure of 1000kN/m^2. Research on flat decks [58] has shown 700kN/m^2 as a maximum but in the knowledge that most of it could be lost over the first few months, values of 1000 to 2000 have also been proposed [78]. Earlier tests on 2.1m span arches, Section 6.8, established that only minimal tensions will provide sufficient friction in an arch, so the lower level of 1000kN/m^2 was chosen as sufficient. The bars passed through the laminates as close as possible to their centres while still allowing a curve to be formed. The moisture content, species and preservative treatment of the timbers all have a significant effect on the tensions necessary to maintain friction. The moisture content was 18%, in accordance with the requirements described in Section 5.2.1, Chapter 5 and the laminates were allowed to dry after preservative treatment. The bars in arches are at closer centres because of the curve and edge distances.

The bridge had been in place for three months before testing. To create a further facet to the testing, only half of the bars which were randomly selected were re-tensioned, to examine the difference it might make in the stiffness characteristics of the bridge. For the subsequent loading, four months later, all of the bars were re-tensioned so that the bridge performance under similar loading conditions could be compared.

6.9.5 Tests and Results

Because of the difficulties of measuring the lateral thrust for a full scale bridge with in situ foundations, and because the results from previous laboratory tests has shown thrusts are as predicted by linear static analysis, they were not measured. Deflection measurements were carried out under increasing loads up to well above design loads. Lateral settlement was measured as the spread of the supports.

Vibration tests were carried out using three independent but similar techniques, all employing accelerometers/transducers to measure the response to a hammer blow, crowds walking over, jumping on the bridge or sandbags being dropped onto the deck. These results were compared to the results from a finite element analysis.

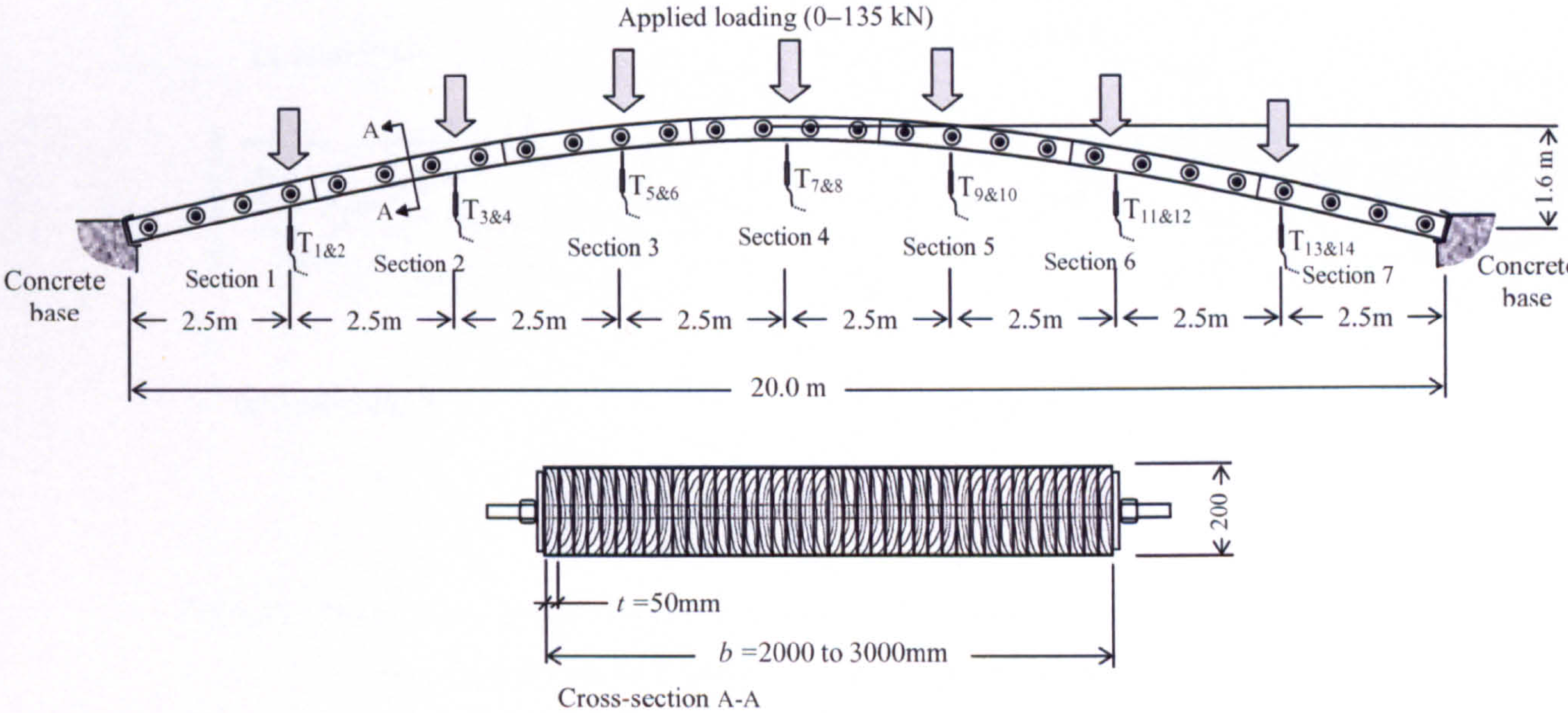
6.9.6 Static Loading

In the first series of tests, displacement transducers were used to measure the deformation profile of the bridge under applied static loadings. Two transducers, one at each side of the bridge, were placed at 2.5m centres along the span of the bridge (fourteen in total). All readings were automatically recorded using a data logging system. As mentioned earlier, the bridge had been in place for three months before testing took place and the stressing bars had relaxed considerably. To create a further facet to the testing, only half of the bars, which were randomly selected, were re-tensioned. This was to examine the difference it might make in the stiffness characteristics of the bridge.

First, 50% of the loading was used to settle and bed down the bridge. The loads were then removed and transducers were adjusted and zeroed. The bridge was subsequently loaded using 9kN bags of sand, placed by the hydraulic arm of the delivery lorry. 15 bags were used to apply 135kN as a UDL, to simulate the 3.2kN/m^2 design load which totals 128kN over the 20m span and 2m wide deck, Figures 6.44 and 6.45, and deflections were recorded at each increment of loading.

Fig 6.44(a) – 20m span UDL loading

Fig 6.44(b) – 20m span UDL loading



Notes: Displacement transducer positions $T_{1\&2}$, $T_{3\&4}$, $T_{4\&6}$ etc for vertical deformation.

Each position represents two transducers – one on either-side.

Fig 6.45 - Details of UDL test.

During unloading, readings were taken after each bag was removed. The results of these tests are illustrated in Figure 6.46 and are compared with the stepwise linear analyses, where at each step, nodal coordinates of the bridge were updated to reflect the new geometry due to deformed profile.

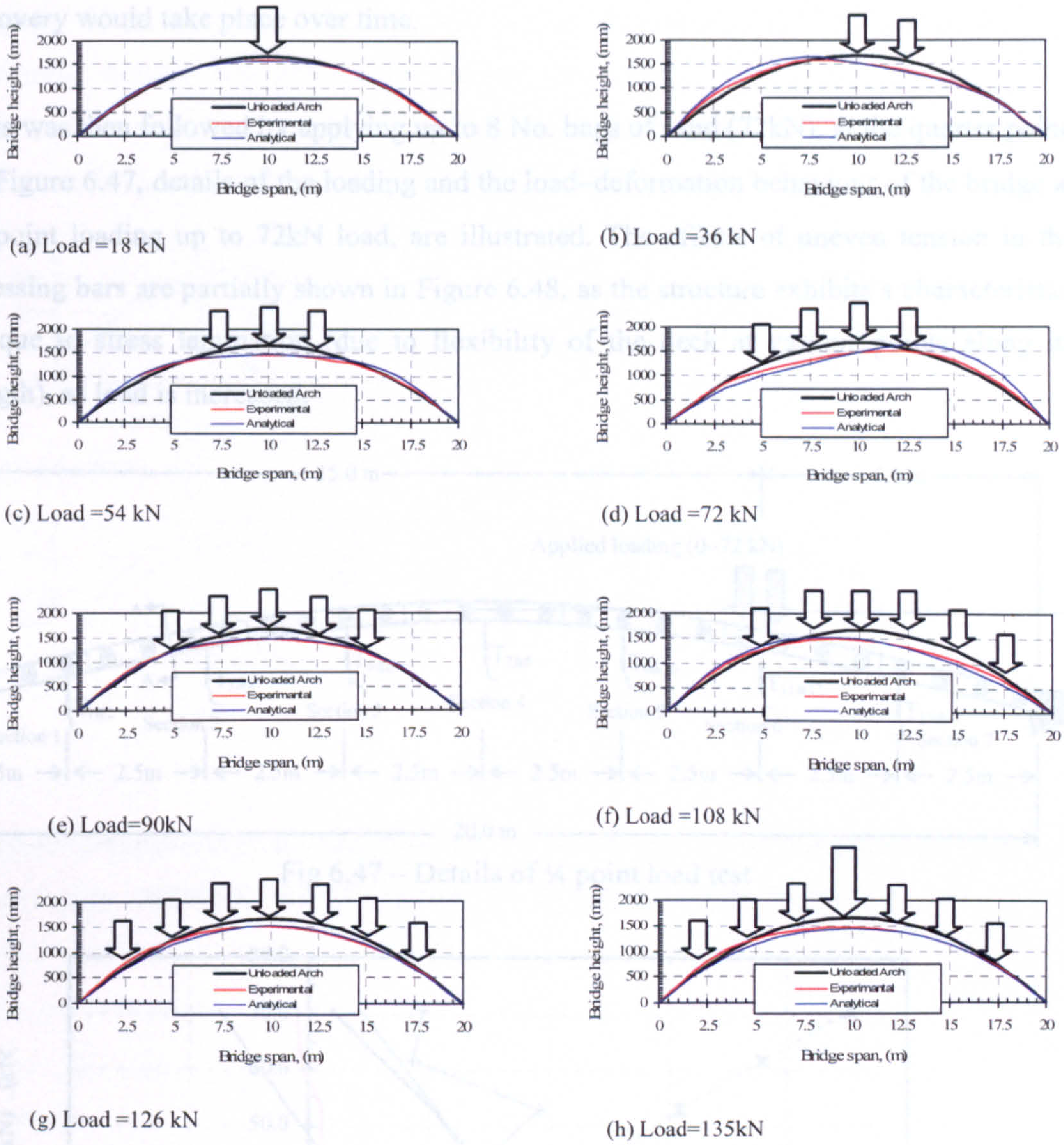


Fig 6.46 - Deformation profile under incremental UDL loading – comparison of analytical and experimental results – deflections are exaggerated for illustration.

The middle 1/3 of the bridge was then loaded with 14 bags (126kN) and subsequently unloaded, using the same procedure. This represented approximately three times design load. The bridge sustained this load with no sign of any distress. On removal of the applied loads, the bridge recovered over 80% of its maximum deflection. Further recovery would take place over time.

This was then followed by applying up to 8 No. bags of sand (72kN), at the quarter point. In Figure 6.47, details of the loading and the load–deformation behaviour of the bridge at ¼ point loading up to 72kN load, are illustrated. The effects of uneven tension in the stressing bars are partially shown in Figure 6.48, as the structure exhibits a characteristic, unique to stress lamination (due to flexibility of the deck at various points along its length), as load is increased.

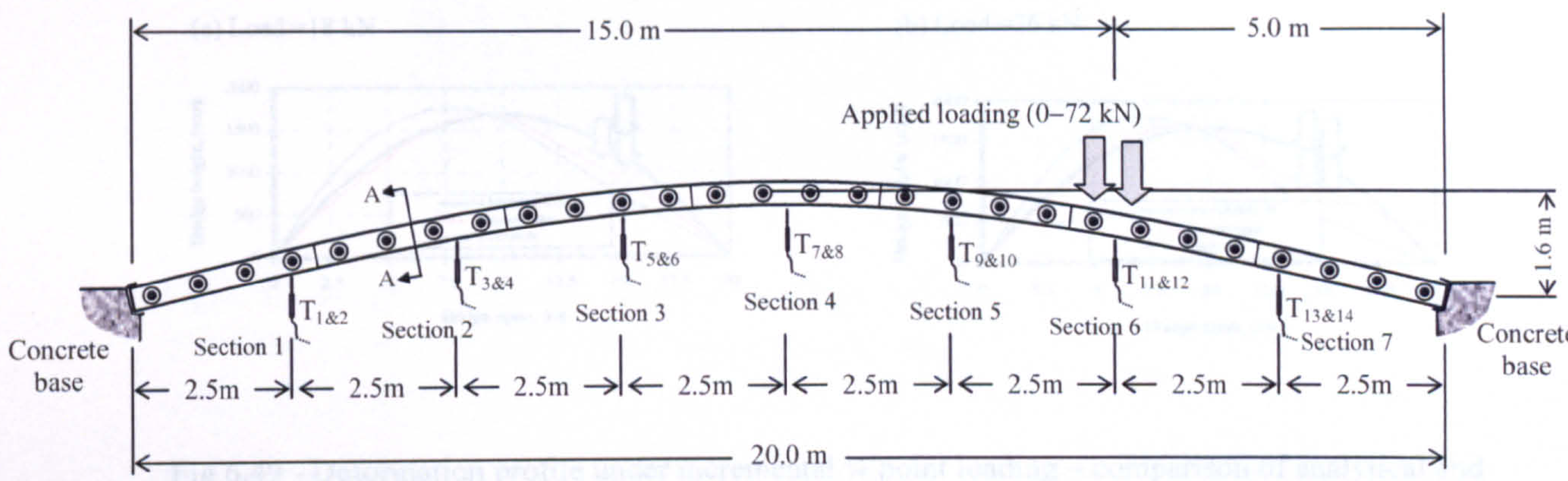
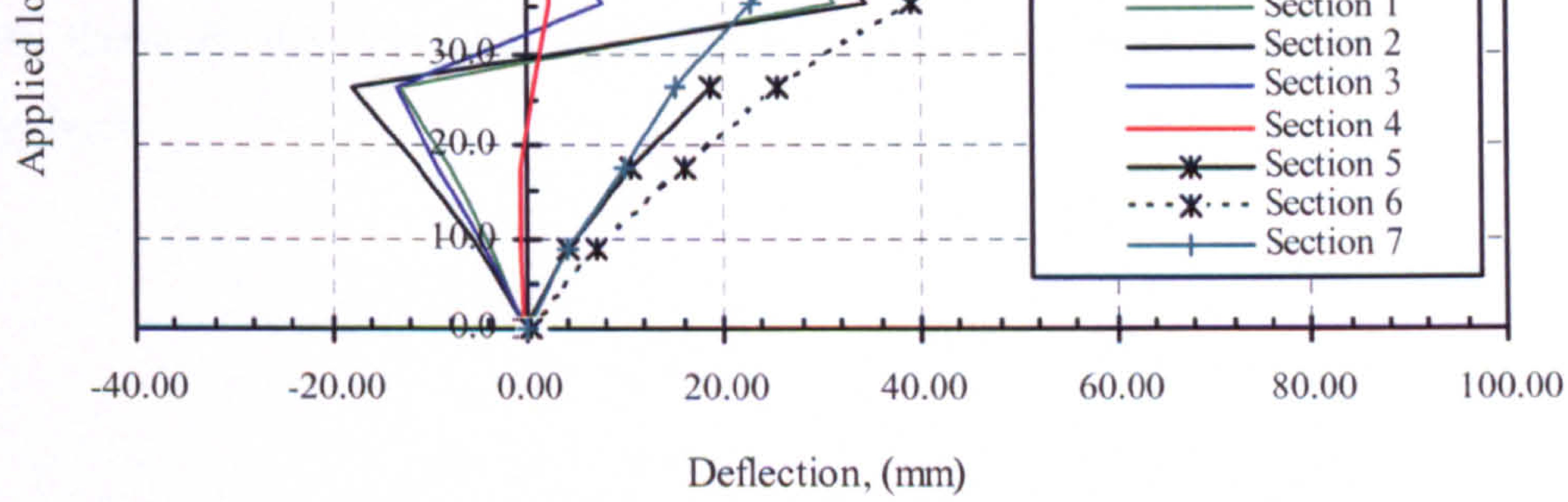


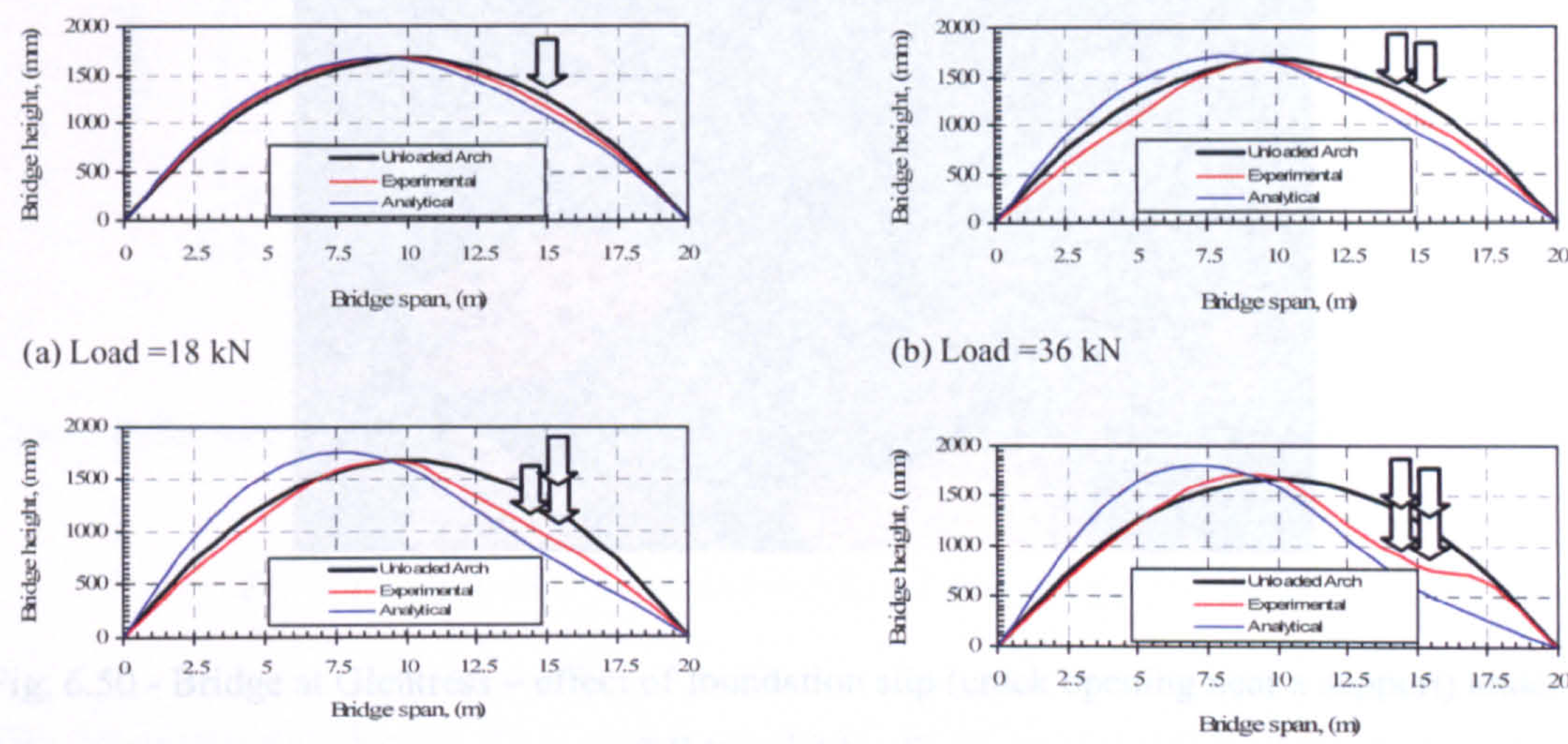
Fig 6.47 – Details of ¼ point load test



Load – deformation when half of the stressing bars were randomly selected and tensioned.

Fig 6.48- Applied ¼ point loading of 0 –72kN using 9kN sandbags.

The results of these tests are further illustrated in Figure 6.49 and are compared with the stepwise linear analyses. With increase in load it was noticed that, due to extreme horizontal thrusts, the small strip concrete foundations started to slip/rotate. The magnitude of support slip at the loaded end was approximately 3mm and at the unloaded end about 5mm both outwards, Figure 6.50.



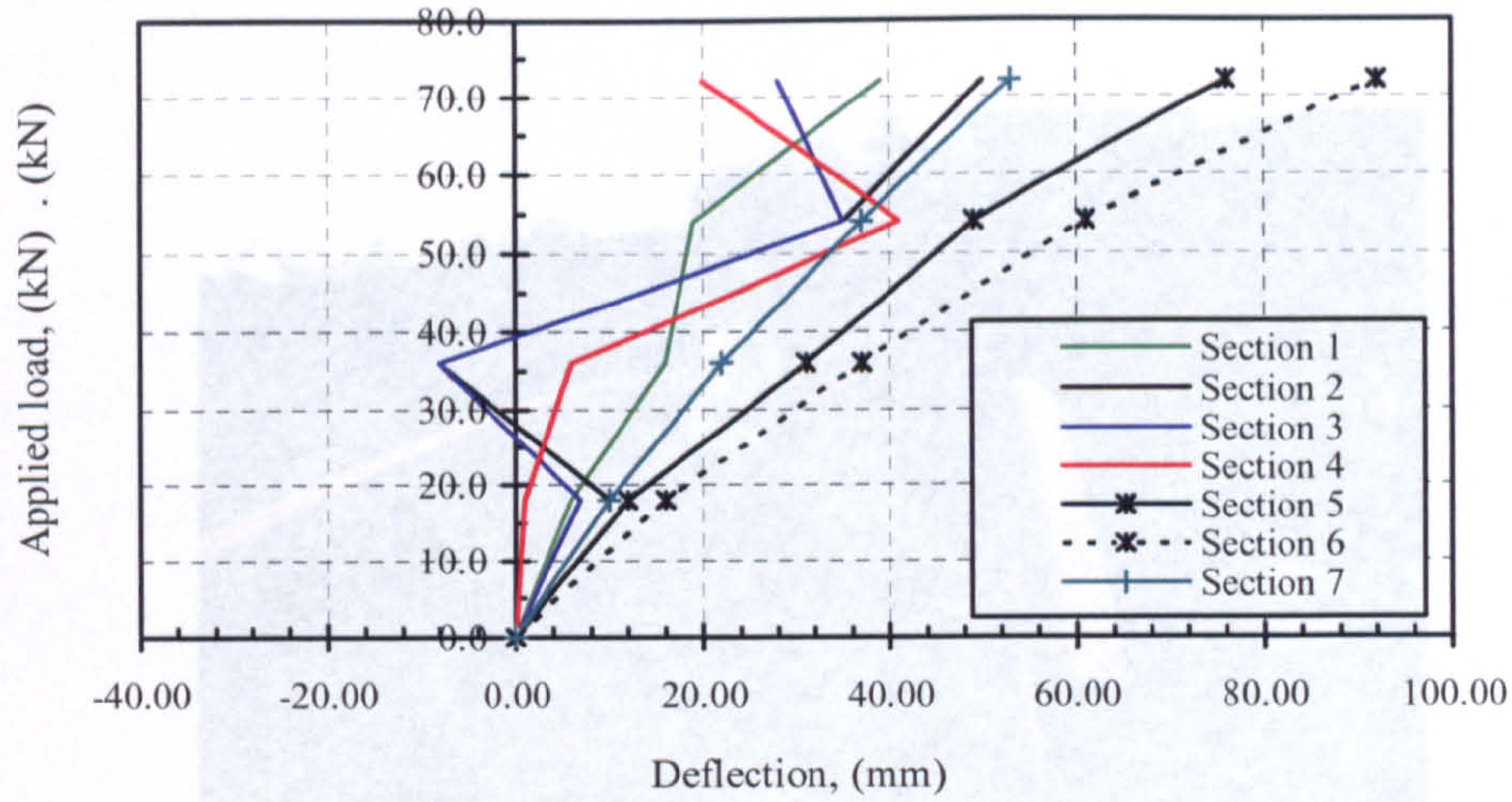
(a) Load =18 kN

(b) Load =36 kN

Fig 6.49 - Deformation profile under incremental ¼ point loading – comparison of analytical and experimental results – deflections are exaggerated for illustration.

Fig. 6.50 - Bridge at Glentress – effect of foundation slip (crack opening near a support) under the full ¼ point loading.

In order to run a check on the most critical loading, and also to test the suitability of a remote deflection measurement equipment for future tests, a further load test was carried out four months later, when the bridge had had time to readjust/recover from the extreme loadings. The quarter point loading, as before, was used. The remote measurement was carried out using a 'SOKIA 4130R3 Total Station', which measures any point in three dimensions to an accuracy of 1mm. For this test, all stressing bars were re-tensioned to 100kN. The results are illustrated in Figure 6.51 and in general gave a good comparison with those in the previous test, Figure 6.48, where only half the bars had been re-tensioned.



Load – deformation when all of the stressing bars were tensioned (4 months later).

Fig 6.51 - applied ¼ point loading of 0 –72kN using 9kN sandbags.

The comparison of Figure 6.48, where only half of the bars are at full tension, and 6.51, where all of the bars are at full tension, further confirmed the previous findings of this programme - that in an arch construction the level of stress in the bars is not as critical as those in the flat bridges [76]. This is clearly shown by the load deflection curves for Section 6 (Figure 6.51), the point under the load. They are the same in both cases even with different tensioning characteristics.

6.9.7 Dynamic Loading

Fig 6.52 - Bridge at Glentress during dynamic testing – excitation by crowd walking.
vibration tests

The first set of vibration tests was carried out using four vertical and two horizontal Pinocchio Vibraphones connected to an 8-channel TEAC LX10 data recorder, in conjunction with ARTeMIS test planner and modal analyses software. The excitation required was provided by crowd walking steadily over the bridge, Figure 6.52. The measurements were made in the sequence shown in Figure 6.53. Dashed arrows indicate free moving sensors, and solid arrows indicate reference sensors.

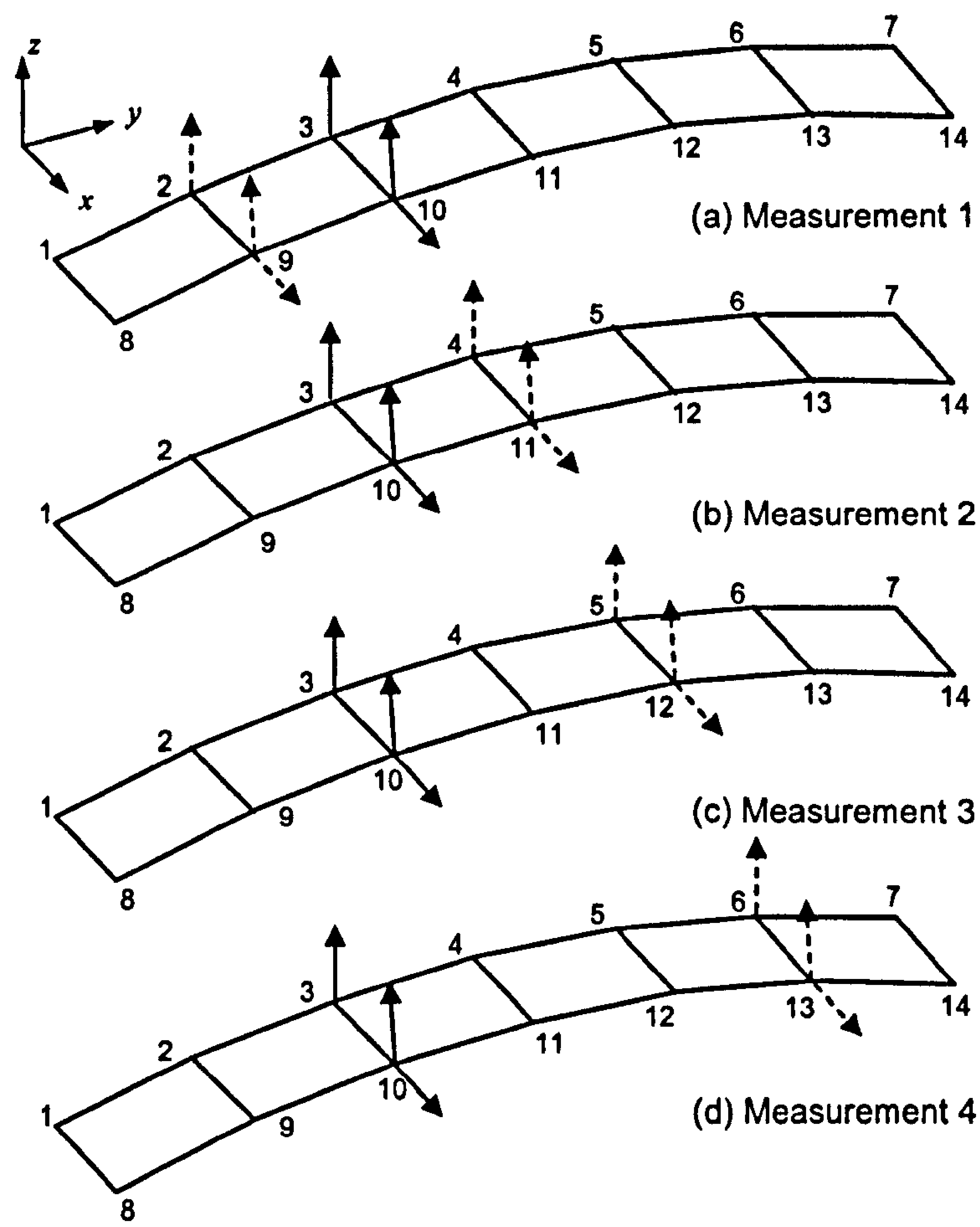


Fig 6.53 - Bridge geometry – data set measurements.

In Table 6.3 the number of records used, the number of samples in each record for modal analysis and the Nyquist frequency are detailed.

Table 6.3 - Acquired data from ARTeMIS dynamic test

No. of Records (DOFs)	6
No. of Samples in Each Record	5000
Duration of Each Record	97.66 s
Nyquist Frequency	25.6 Hz

In ARTeMIS Analyser, the data was processed with a default signal processing configuration, including a 1024-lines spectral density estimation. Figure 6.54 shows the singular values of the spectral densities of the third measurement. During the measurements, modal analyses were made using the fast Frequency Domain Decomposition Peak Picking technique. This was to provide quality checking of the data together with verification of the sensors and their positions.

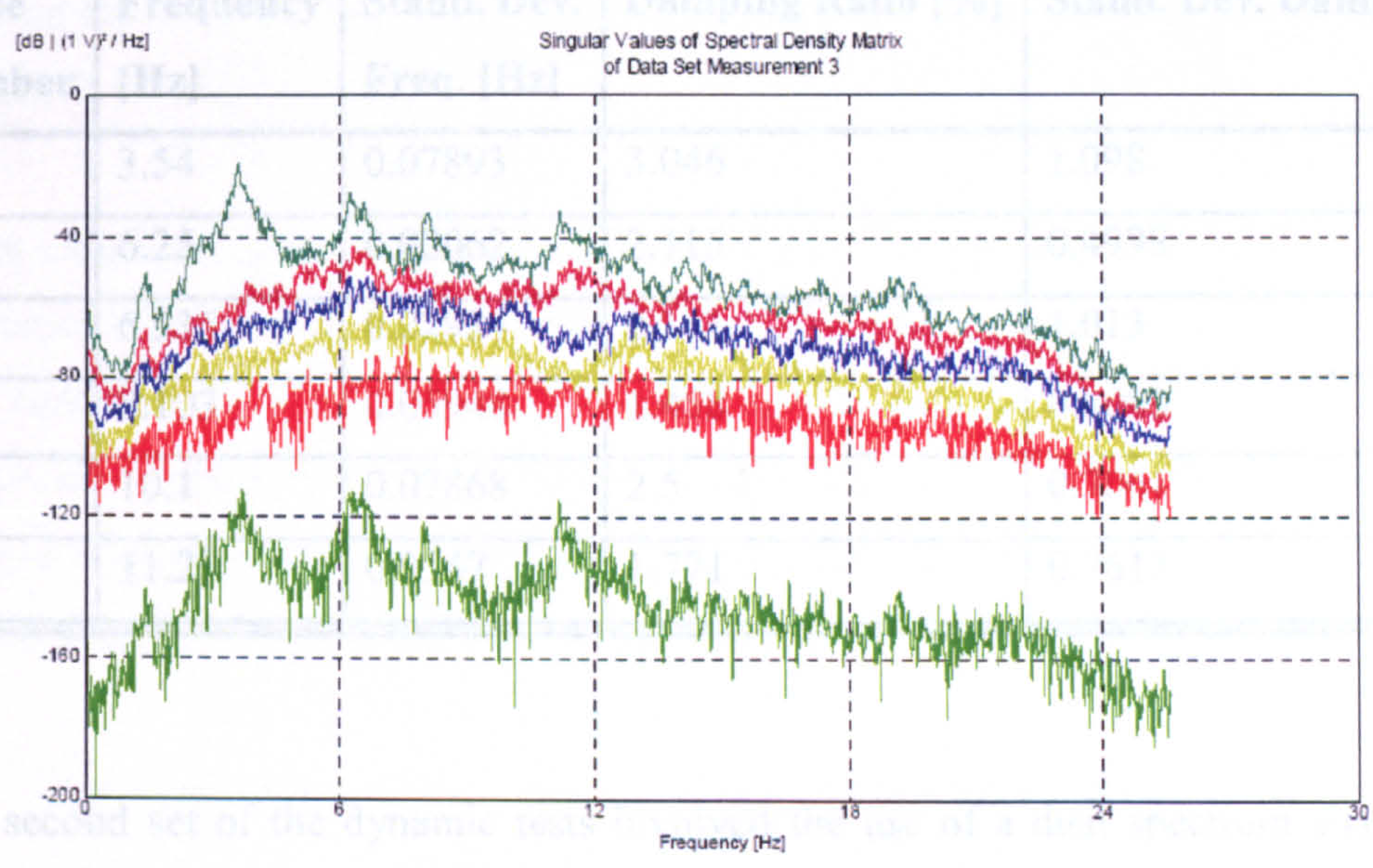


Fig 6.54(a) Singular values of the spectral densities

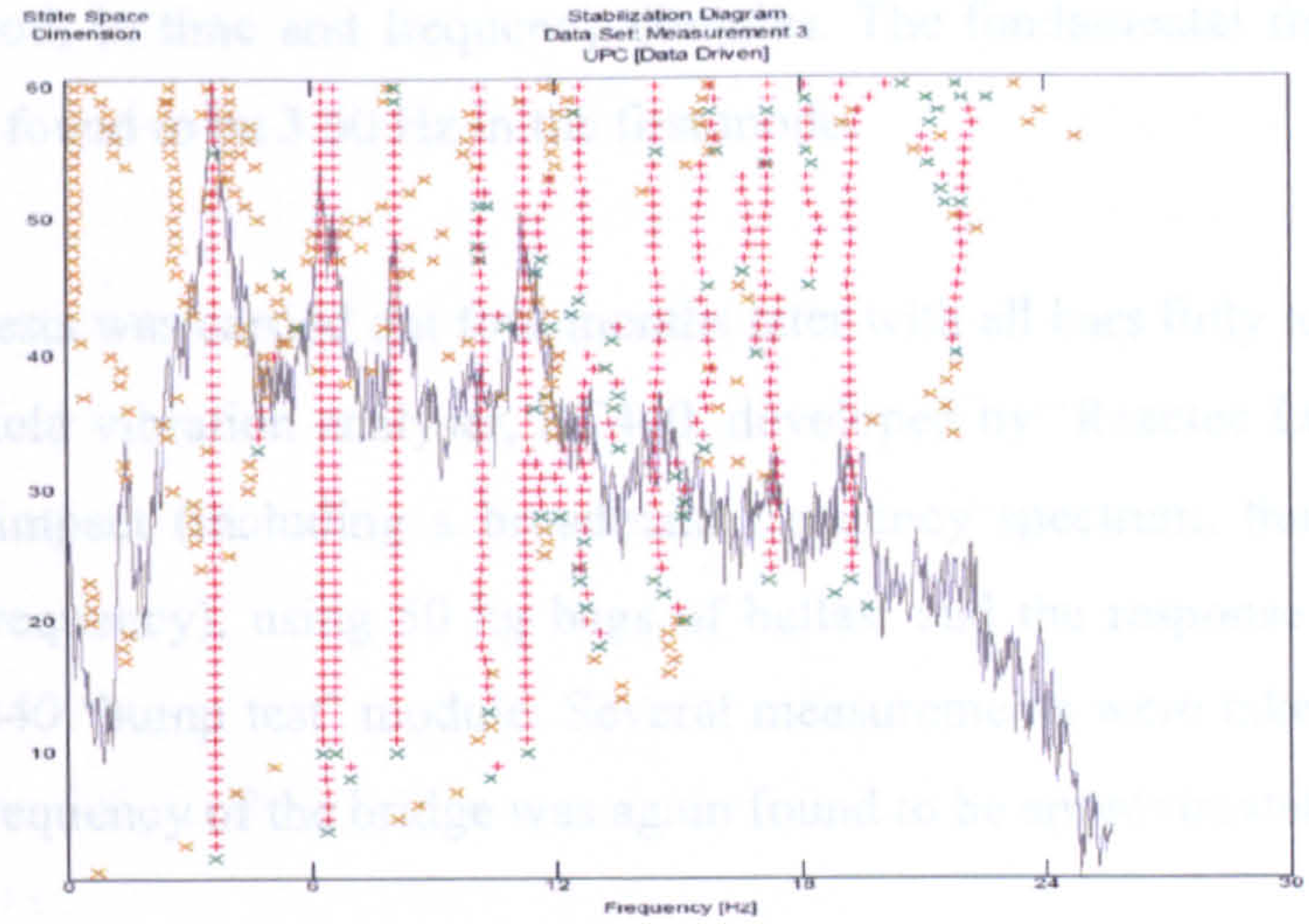


Fig 6.54(b) Stabilization diagram

Fig 6.54 - Dynamic response – sample output for measurement 3.

The first six modes were found and are shown in Table 6.4.

Table 6.4 - First 6 modes estimated using the UPC, Stochastic Subspace Identification technique.

Mode Number	Frequency [Hz]	Stand. Dev. Freq. [Hz]	Damping Ratio [%]	Stand. Dev. Damp. [%]
1	3.54	0.07893	3.046	1.098
2	6.25	0.02062	2.115	0.4998
3	6.535	0.02934	2.111	1.013
4	8.103	0.05948	1.458	0.6083
5	10.1	0.07868	2.5	0.7061
6	11.29	0.0547	1.771	0.7613

The second set of the dynamic tests involved the use of a dual spectrum analyser and impact excitation method. The impact hammer was used to excite the structure. The response was recorded using two accelerometers. Both the excitation force and the response signals were recorded, using a multi-channel spectrum analyser. The results were analysed, both in time and frequency domains. The fundamental frequency, using this method, was found to be 3.60 Hz in the first mode.

The third set of tests was carried out four months later with all bars fully tensioned, using a versatile handheld vibration analyser, RT440, developed by 'Reactec Ltd'. The bridge was excited by impact (including a broadband frequency spectrum, thus exciting any system natural frequency), using 50 kg bags of ballast and the response was measured utilising the RT440 'bump test' module. Several measurements were taken at both sides and the natural frequency of the bridge was again found to be approximately 3.5 Hz in the first mode.

The experimental results obtained from the three different methods (including both input and instrumentation), compared very well with each other, indicating that the fundamental natural frequency of the bridge, without any topping of bitumen macadam and backfill at the abutments to reduce slope for access, is approximately 3.5 Hz. As mentioned earlier, a simplified finite element analysis carried out to estimate the natural frequency of the bridge - assuming it as a single mass of homogeneous material and ignoring the laminate slip and the flexibility within the composite mass of the deck - was just above 4.0 Hz.

These values are close to the frequency which can be applied by vandals (2.5Hz), but resonance is unlikely to occur because ten persons could never impart sufficient energy to an 8 tonne structure at 1Hz over the optimum that they can normally impart. At these values, this parameter could be considered critical in the design of a footbridge but, with a dense bitumen macadam topping as waterproofing, the FNF will increase beyond the critical zone.

6.10 6m Test Arches – Flat - 250mm Rise – 500mm Rise – 1000mm Rise

The first trial bridge of 6m span was a half scale of the 12m first design, but the laminates were 50mm and therefore not to scale. The 2.1m span laboratory test bridges were considered too short and stiff to be fully representative of the scale of bridges which are now known to be necessary for commercial footbridge loading. The field tests had shown correlation between load and scale. It was therefore justified to undertake another set of laboratory tests to a true scale, and large enough to avoid boundary effects. It was therefore decided to load test four 6m span bridges with 25mm wide laminates, and each 475mm wide. The results from these tests were considered vital to consolidate the previous findings.

Three different rises (250mm, 500mm and 1000mm) were chosen and two flat bridges. The focus of the tests was to accurately measure parameters which were now known to be of special importance. The first tests concentrated on showing arch action and correlation with elastic analysis. It had become clear that lateral movement and thrusts of a well scaled model must be measured to give a true picture of all of the structural actions.

Figure 6.55 shows a flat span with four point loading. Figures 6.56, 6.57 and 6.58 show the load tests and the set of arches with the different loading regimes.

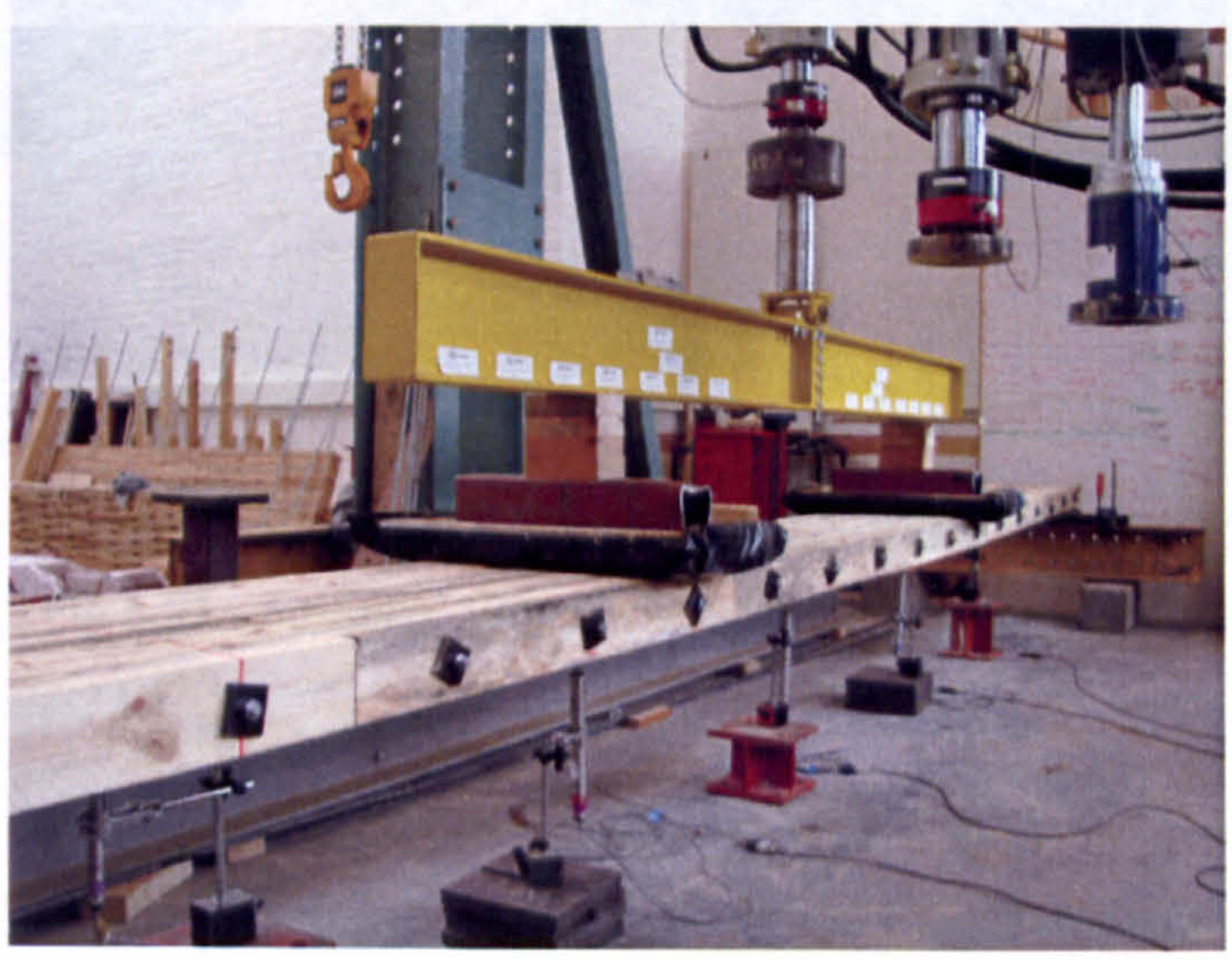


Fig 6.55 – Flat 6m span under four point loading

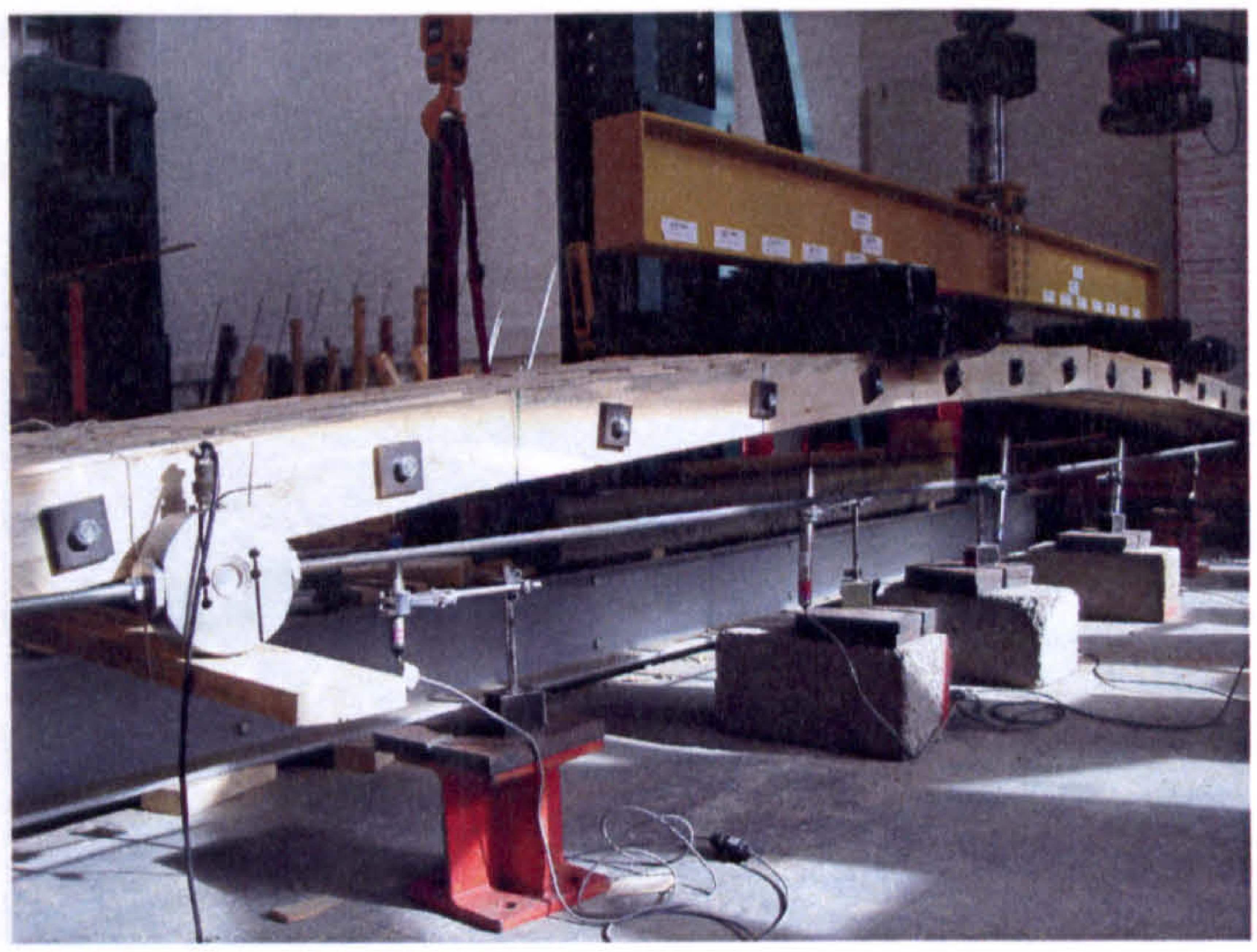


Fig 6.56 – 6m span, 250mm rise under four point loading

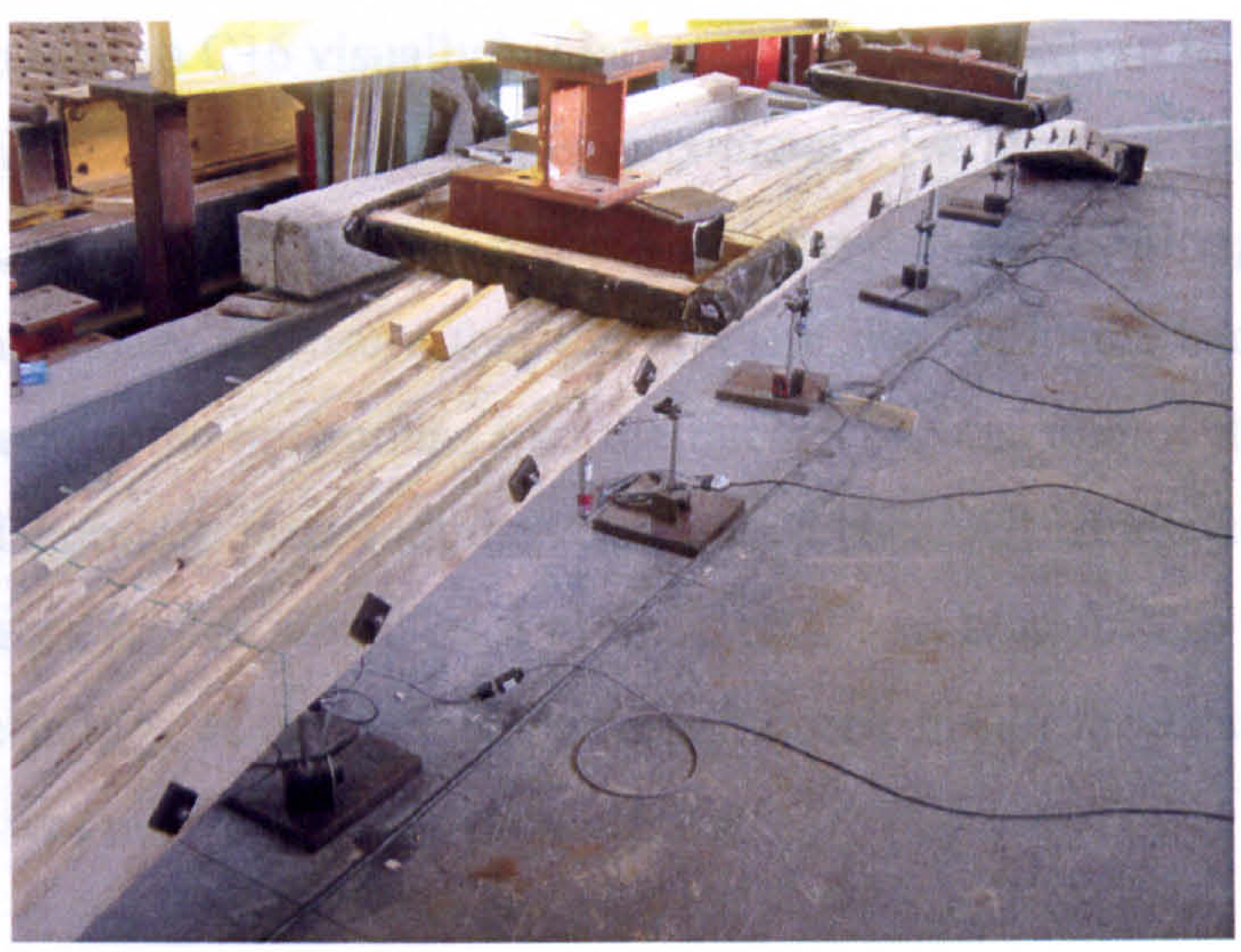


Fig 6.57 – 6m span, 500mm rise under four point loading

Fig 6.58 – 6m span, 1000mm rise under quarter point loading

6.10.1 Preparation of Material for Arches

The timber was graded as C16 visually but later tests were carried out to establish the true modulus of elasticity. The timbers were approximately 1m long for the flat bridge, the 250mm rise arch, the 500mm rise but had to be reduced to 500mm long for the 1m rise arch. This was necessary to ensure that the holes in the laminates had sufficient edge distance and to reduce the projections into the deck. This was one of the improvements made over early laboratory tests with the 2.1m span arches. All laminates had three holes so the effective cross section would be ⅔ of the actual cross section. The moisture content was approximately 15% as the timber had been stored inside the laboratory for some time. The timber was not treated with preservative.

The tensioning was carried out with 12mm diameter threaded bars as had been used for the 2.1m spans. The same method described in Appendix 4 was used to determine the tension from a torque setting. Again this method was chosen to facilitate the changes in tension which was a major factor in the tests to evaluate the minimum tension necessary for adequate lateral load transfer. Because larger tensions were required the threads on the bars eventually began to fatigue. Future tests of this kind will use a larger diameter or higher grade steel. The loads were spread into the timbers by a square washer plate which worked adequately for most tests but because the outer laminates were softwood there was some bearing failure.

6.10.2 Static Load Tests of 6m Span Laboratory Bridges

All of the 6m span bridges were tested using a 'Dartec Modular 9500', as in previous laboratory tests. A preload of approximately 3kN was applied and removed a few times, to eliminate any initial settlements in the arches. The bridges were all subjected to a four-point symmetrical loading condition, as illustrated in Figure 6.59.

They were loaded at a constant rate, up to a little more than their design load, appropriate to each span rise, without causing structural damage. This was a relatively small load for the flat span and much greater for the 1000mm rise arch. The load was then removed and the bridge was allowed to recover. This process was repeated for a number of different lateral tensions, ranging from finger tight to 24kN in each stressing bar.

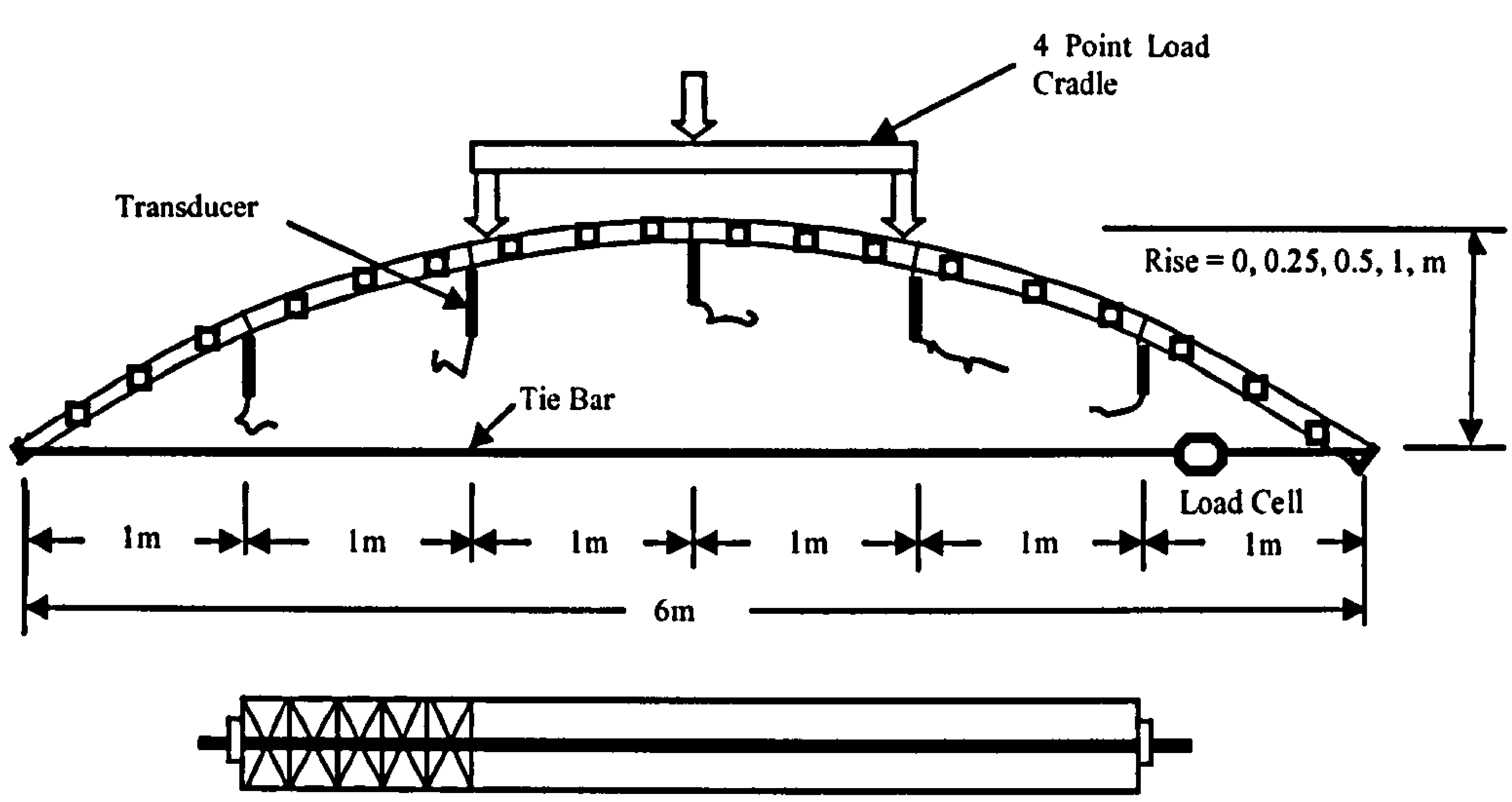


Fig 6.59 - Details of 4 point loading test

The flat spans and three arches were all loaded at their quarter points for various tensions. This is the weakest section of the arch and, therefore, the critical design section.

All of these load tests were within the working strength of the structures, as the primarily aim was to produce results which could be compared to analysis results and different tensions.

The data logger recorded the applied loads and the related deflections, measured by transducers at 1m intervals along the arch and lateral movements of the supports. The lateral thrusts were measured using load cells in the tension bars linking the springings.

6.10.3 Analysis of Results

All of the tests showed similar behaviour to all of the previous tests. However this time longitudinal settlements were measured, lateral tensions were varied and many more readings were taken, so there was more confidence in the findings.

The most significant result from this test programme illustrates that, after reasonable lateral tension is provided, a steady load carrying capacity is generated in the arches. This had been found with the 2.1m spans but was so important it needed confirmation with a larger scale test.

Figures 6.60 to 6.63 show the deflected forms of the flat, 250mm, 500mm and 1000mm rise arches with different lateral tensions to illustrate the effect of variable tension. Four different tensions, zero, 7kN, 12kN and 20kN or 24kN were applied while gradual loading was applied from zero up to 20kN or less if the deflection became too great. The flat span is shown at only 1kN load because, at low lateral tension the span could not take any more without collapsing. However with full tension it reacted in a similar way to the 250mm rise arch. Figures 6.60, 6.61, 6.62 and 6.63 show deflected forms for each arch with the variable tension. The flat span deflections are shown unfactored, the 250mm rise was multiplied by two to exaggerate the deflection while the 500mm and 1000mm rises were exaggerated by five.

These plots are shown to illustrate the marked difference made by the tension and the significant increase in load capacity between flat span and arches. Figure 6.64 shows a plot of load against deflection for each arch at full lateral tension to illustrate the significant increase in load capacity.

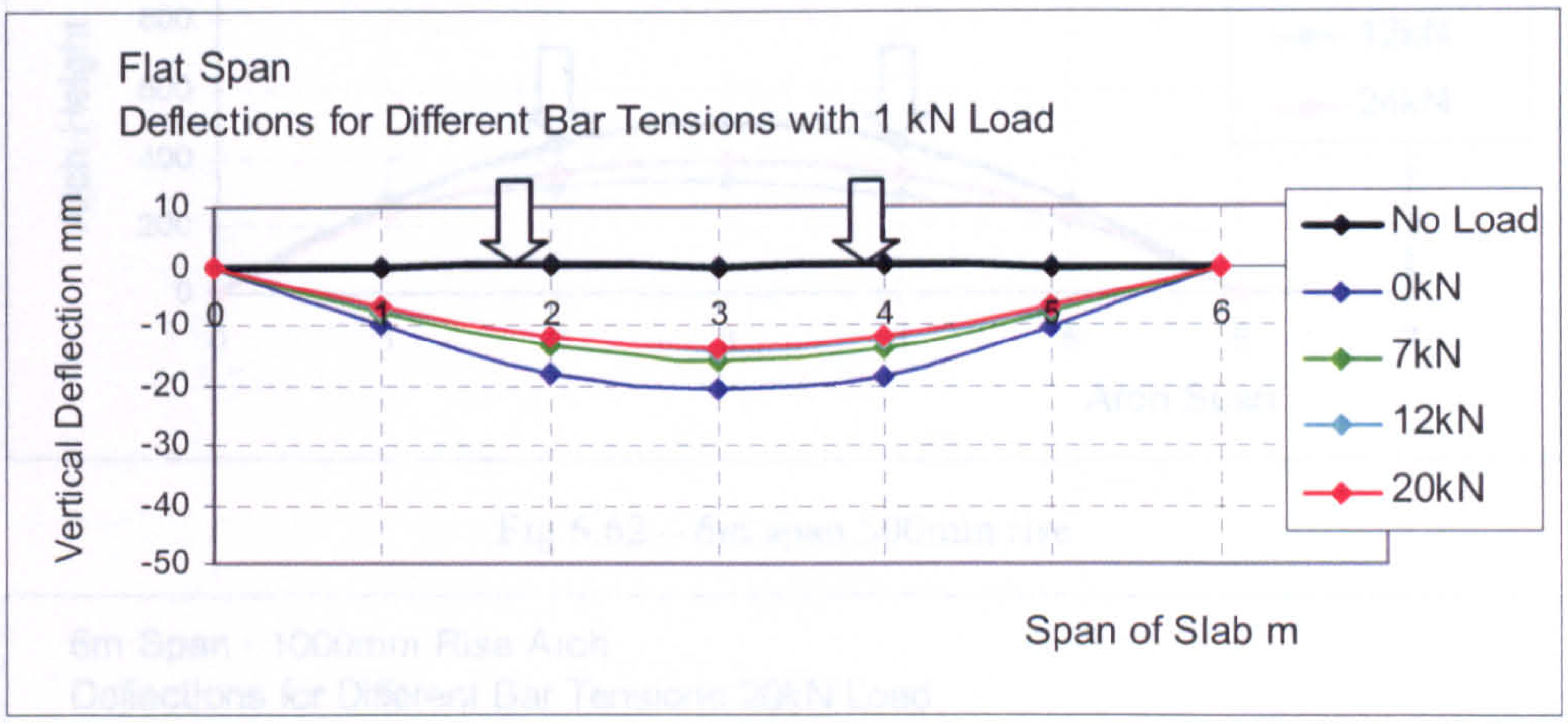
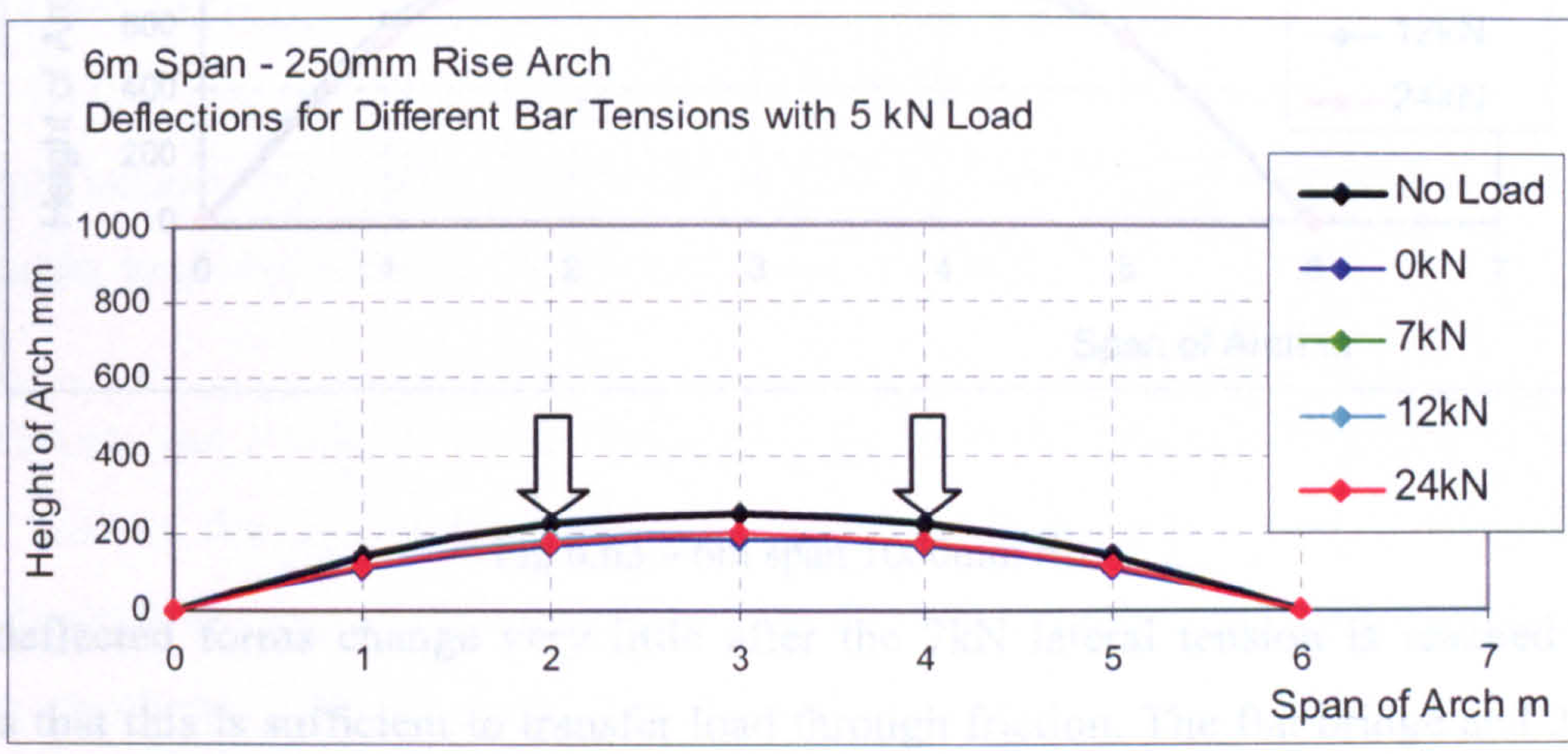


Fig 6.60 – 6m span flat bridge



Fog 6.61 – 6m span 250mm rise

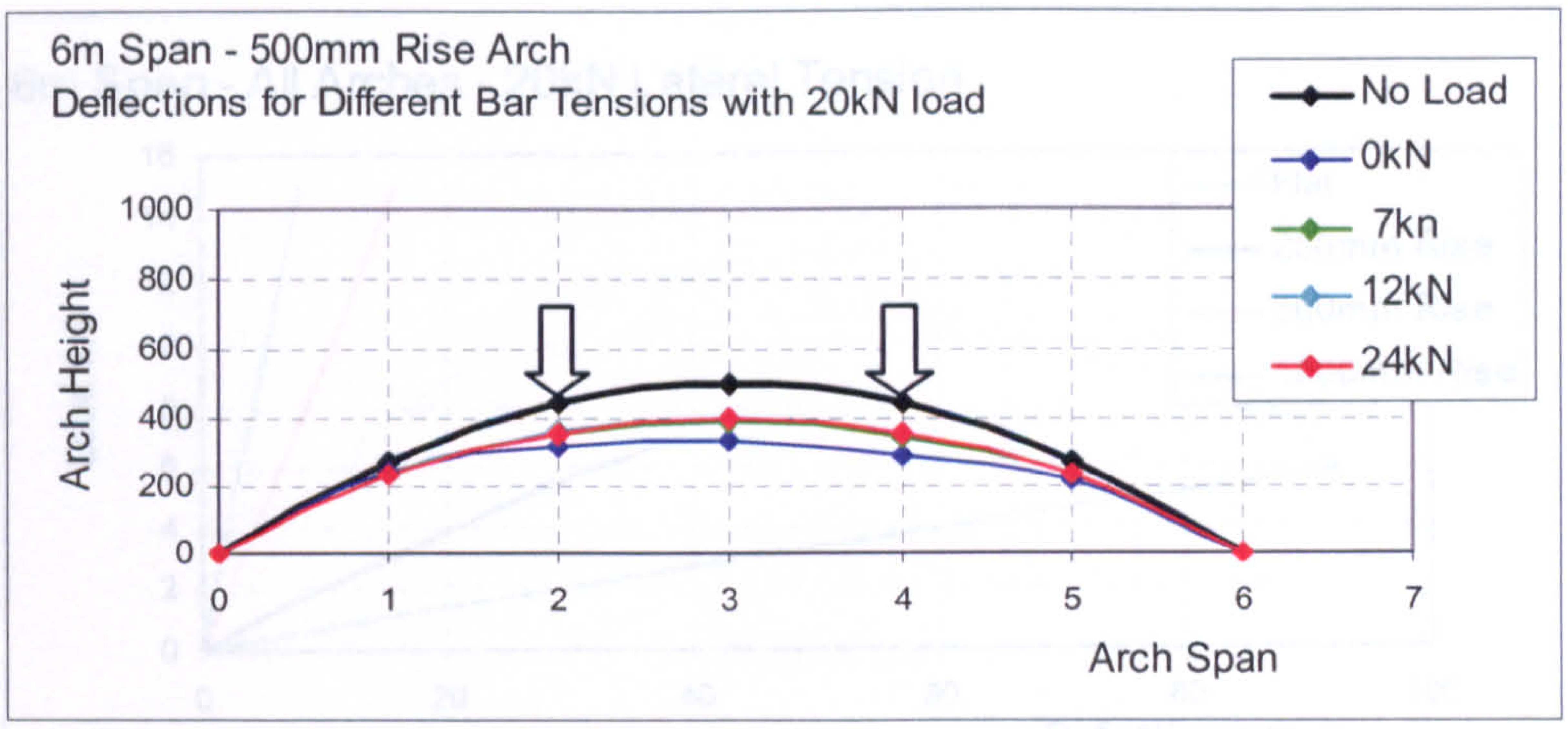


Fig 6.62 – 6m span 500mm rise

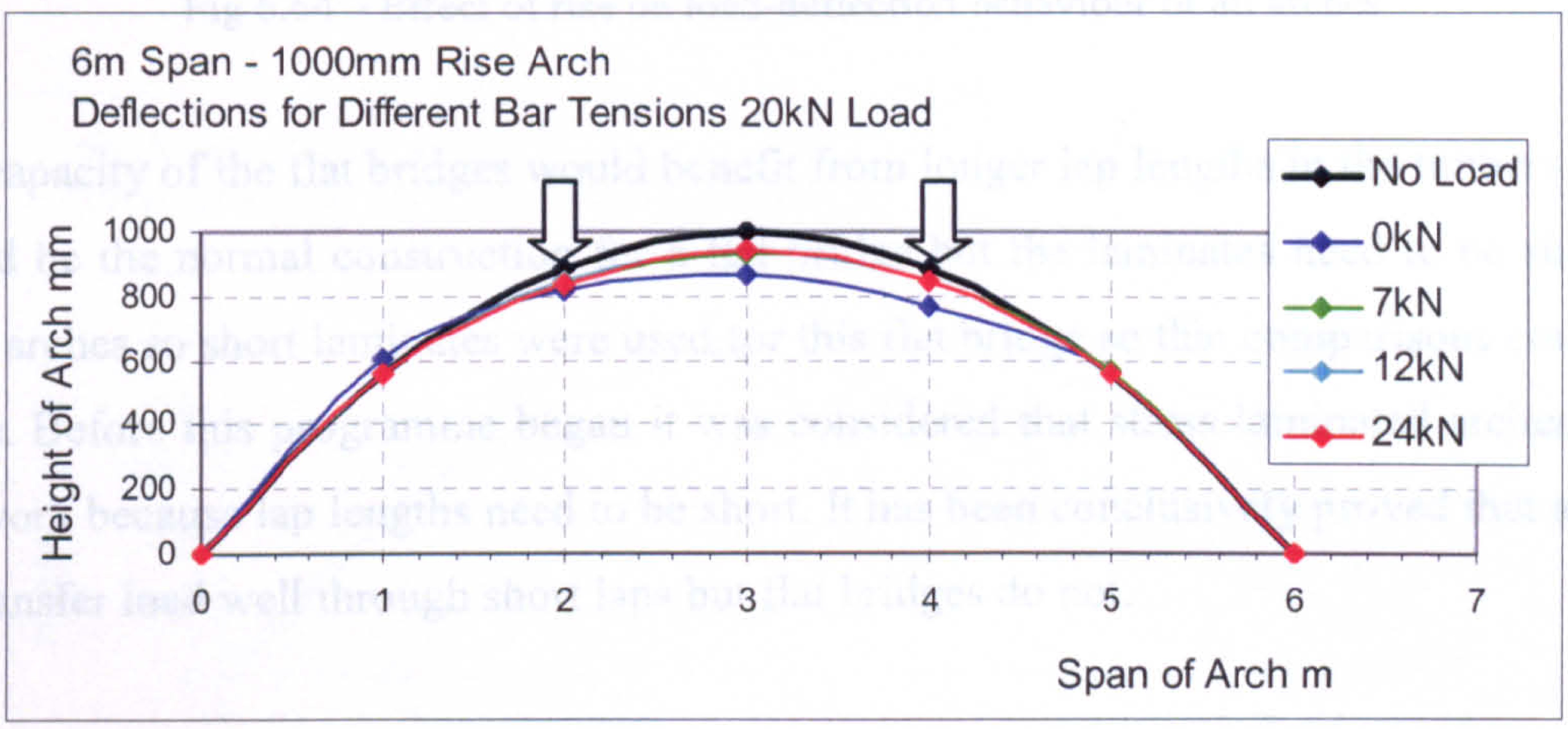


Fig 6.63 – 6m span 1000mm rise

The deflected forms change very little after the 7kN lateral tension is reached which shows that this is sufficient to transfer load through friction. The flat bridge and 250mm rise arch took very little load which demonstrates the load capacity of an arch.

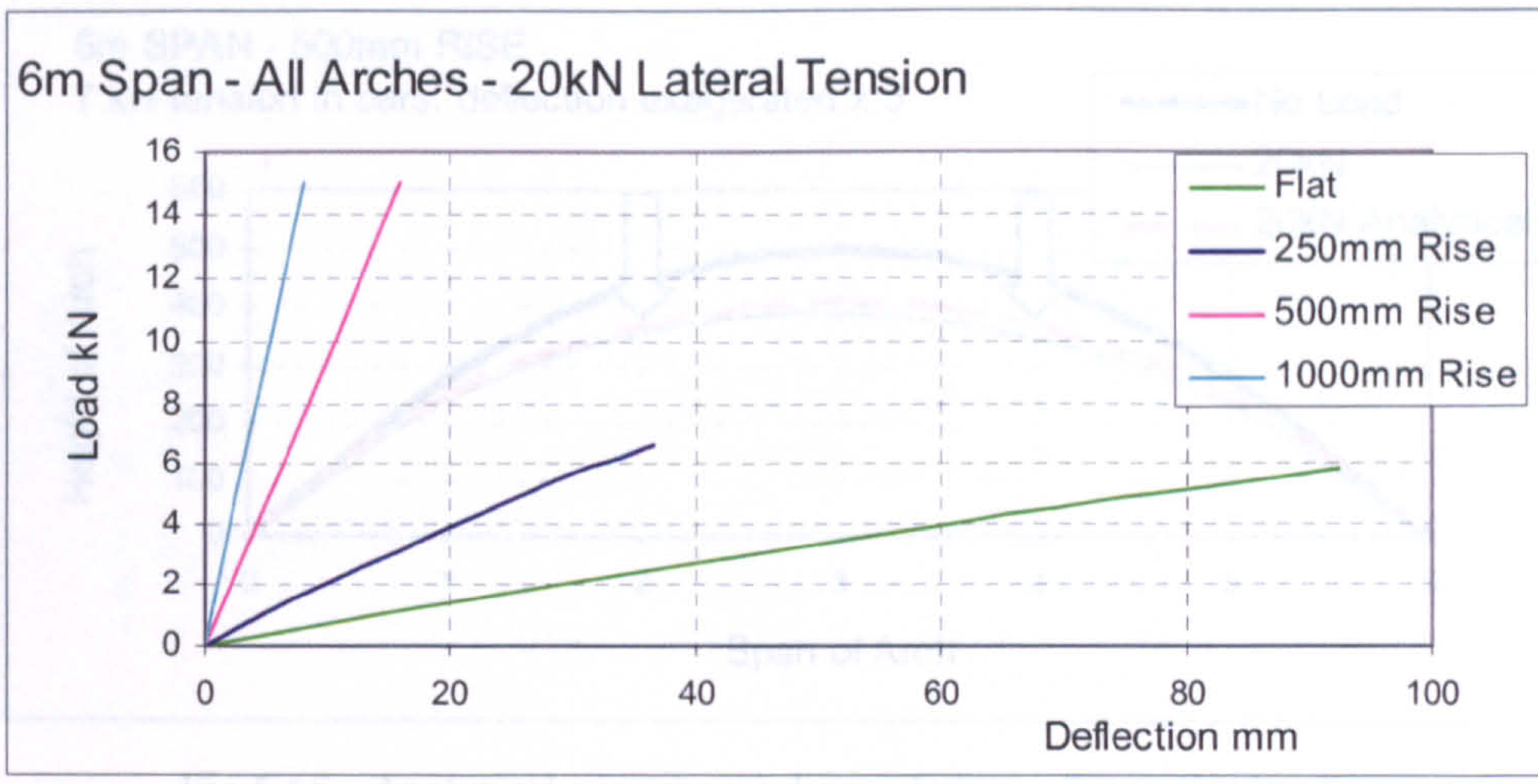


Fig 6.64 – Effect of rise on load-deflection behaviour of all arches

The capacity of the flat bridges would benefit from longer lap lengths in the timbers. This would be the normal construction for a flat bridge but the laminates need to be short to form arches so short laminates were used for this flat bridge so that comparisons could be made. Before this programme began it was considered that stress laminated arches may not work because lap lengths need to be short. It has been conclusively proved that arches do transfer load well through short laps but flat bridges do not.

The 500mm and 1000mm rise arches were both loaded to approximately 20kN, which is about half of the failure load of the original 6m span arch tested three years ago (Freedman & Kermani ICE 2004 [76]). This again shows that, within the working limits of the structures, they act elastically and their behaviour is predicted by linear elastic analysis.

Figure 6.65 shows the 500mm arch with central loading and Figure 6.66 with quarter point loading. These are examples to illustrate the correlation with elastic analysis. The full array of all four structures with the six loading conditions at two different lateral tensions, zero and full, are shown in Chapter 7.

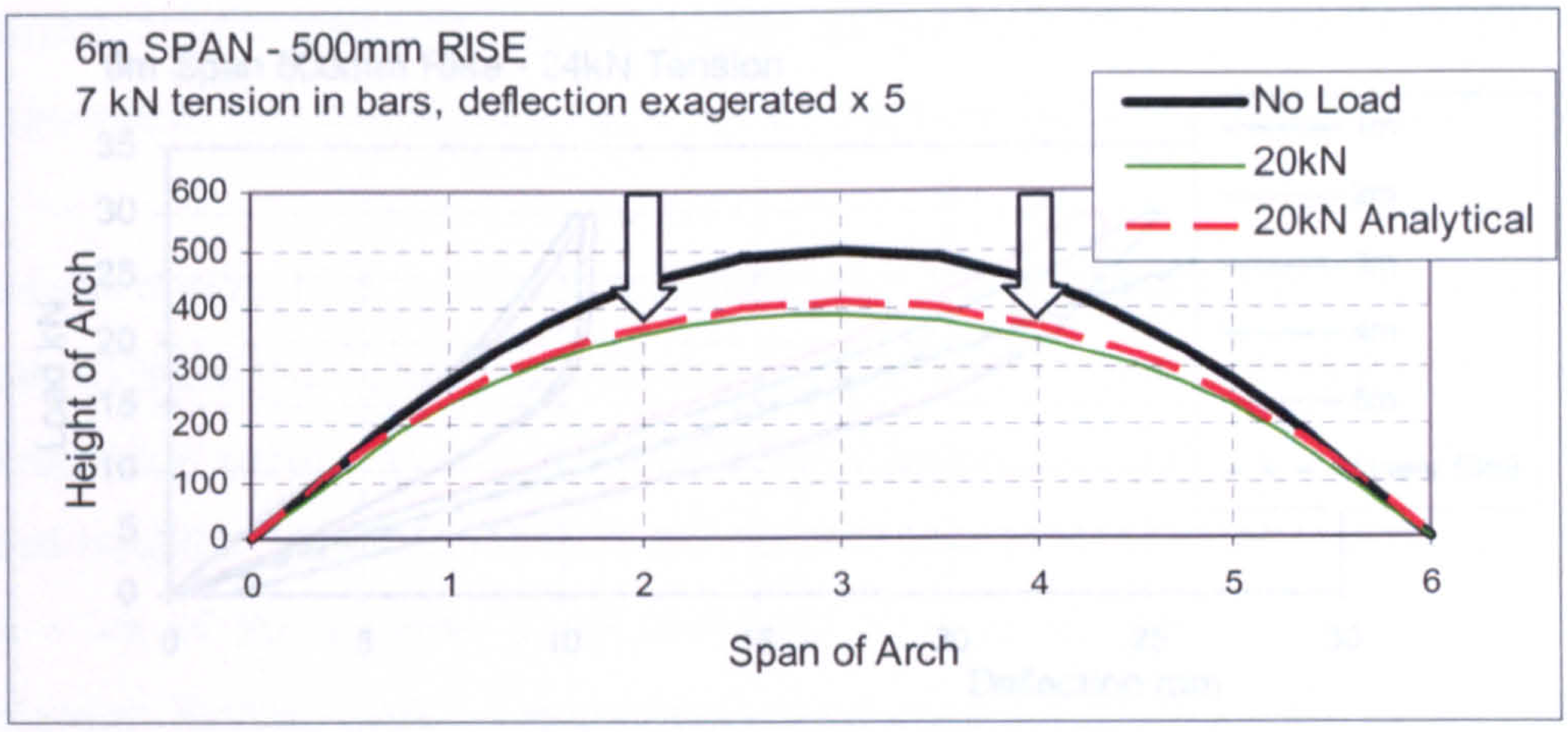


Fig 6.65 - Analytical experimental correlation – four point loading

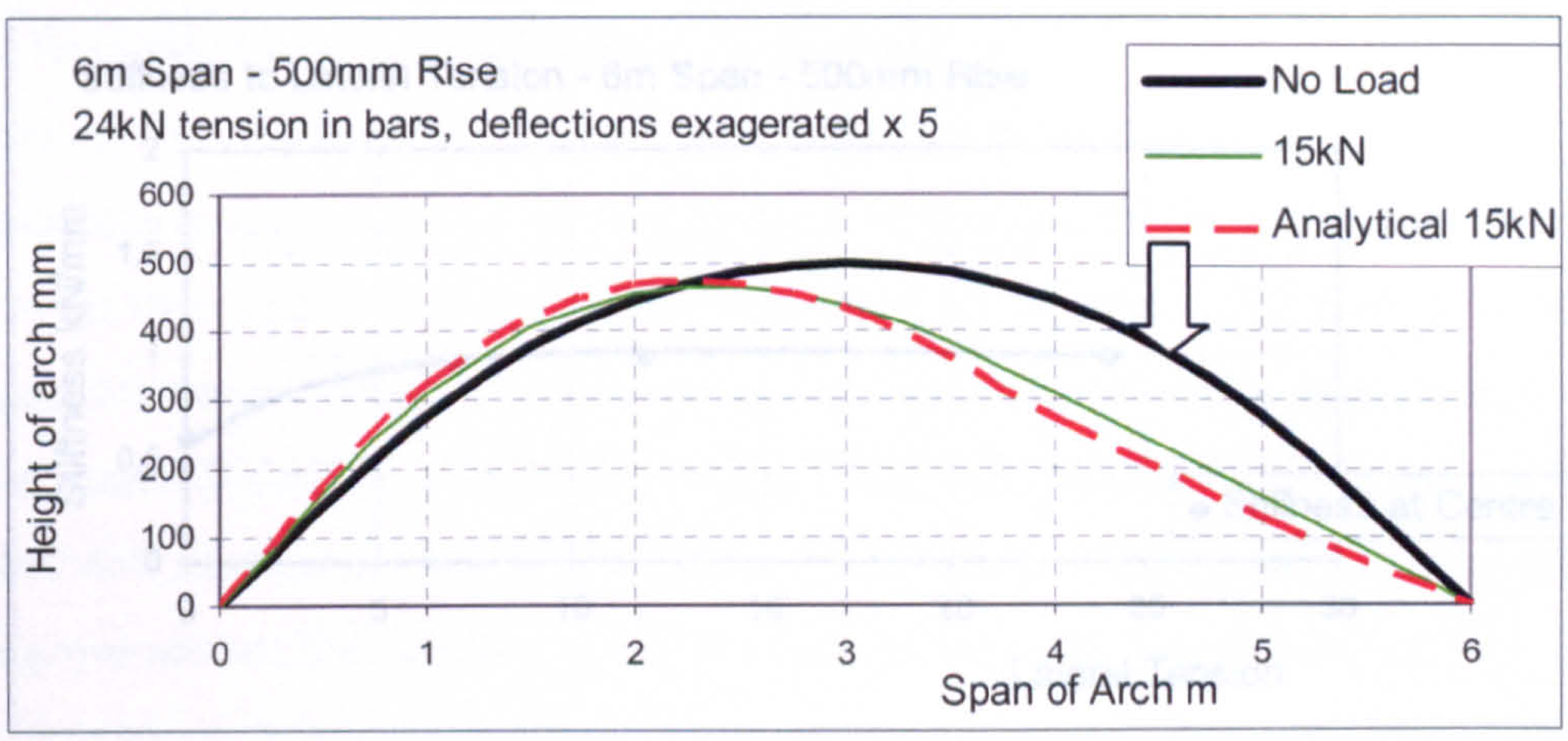


Fig 6.66 Analytical experimental correlation – quarter point load

Figure 6.67 shows the load deflection plots, for central loading on a 500mm rise, for positions at each 1m point along the bridge during loading and unloading. This example is for full lateral tension. A trendline is shown for the central point. It will be shown in Chapter 7 that the gradient of this line is a measure of stiffness. Figure 6.68 shows the stiffnesses plotted against lateral tension which illustrates that full load bearing capacity is reached at 7kN.

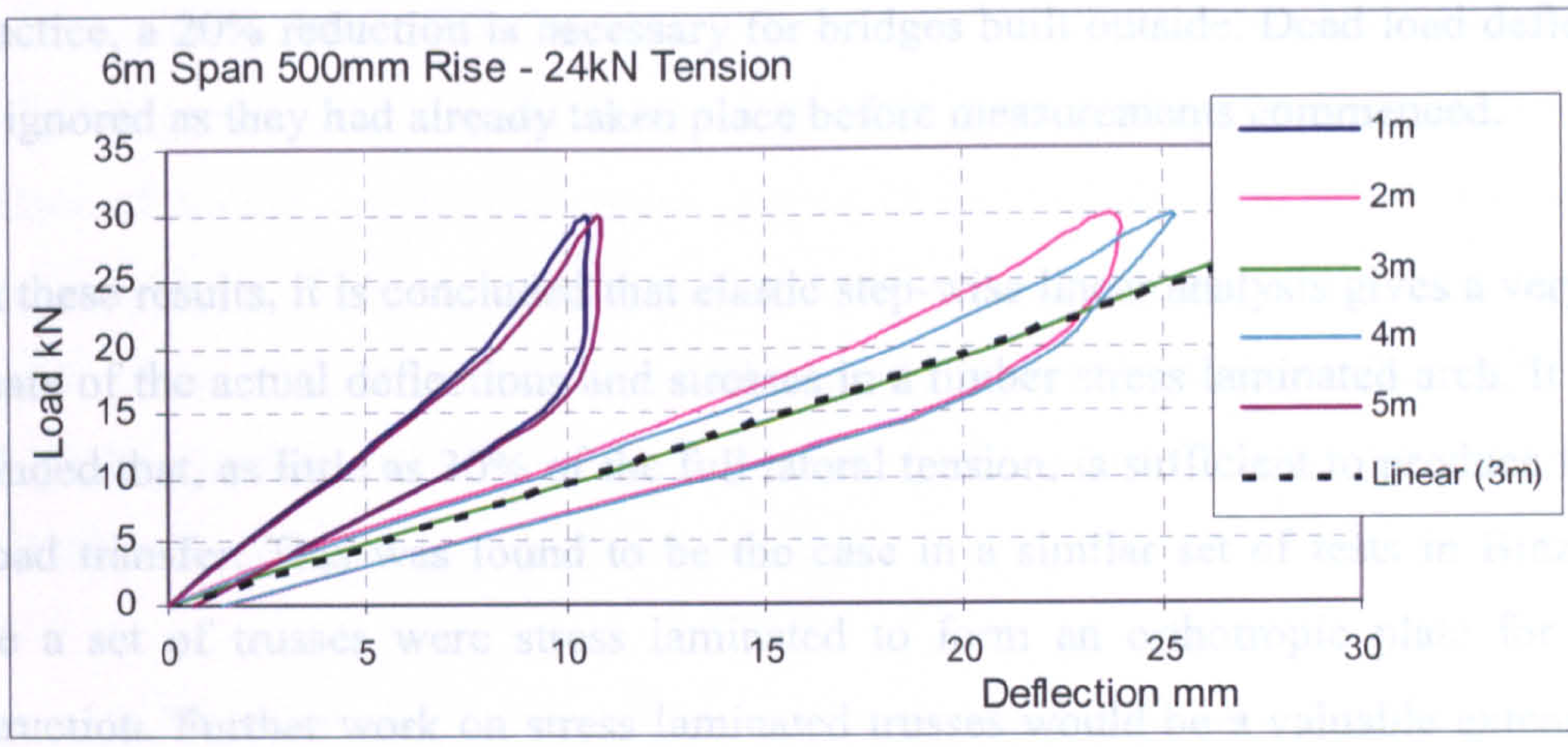


Fig 6.67 – Load deflections at 1m intervals

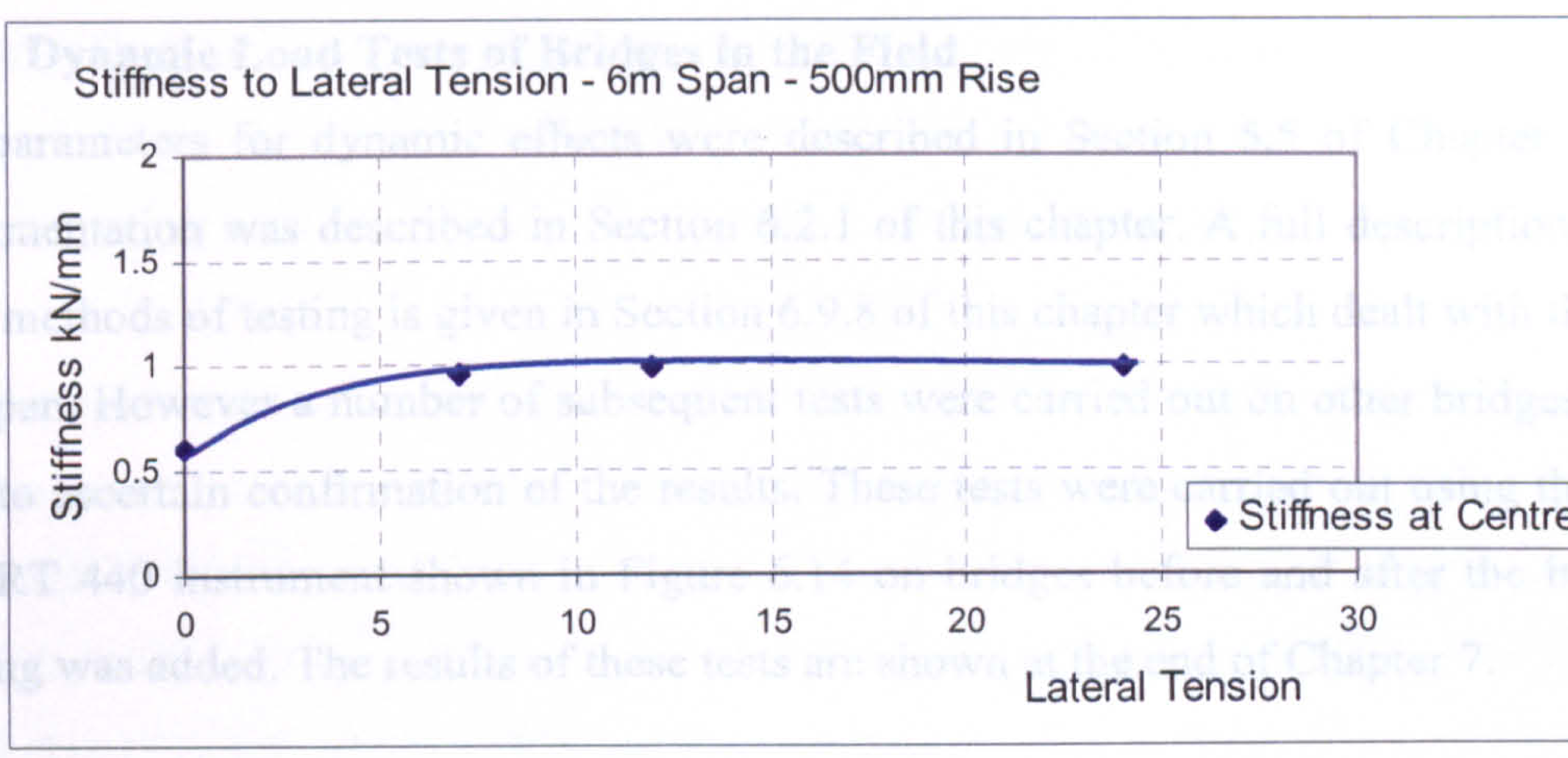


Fig 6.68 – Stiffness to lateral tension – 6m span 500mm rise

The elastic analyses used to make the comparisons in Figures 6.65 and 6.66 took account of the support movement which occurred in the test at the particular loading and tension. This was found to be critical for close correlation in these tests but even more so in the 20m span test bridge [79] (Kermani & Freedman ICE Nov 2005) where movement was greater. The discontinuities in the deck, at the butt joints, were accounted for by reducing the cross section to twelve timbers in the analysis. This value depends on how many tension bars are provided in each laminate. In these tests there were three, hence the 1/3 reduction in cross sectional area. No reduction was taken in the elastic modulus value for exposure because the timber was dry and the tests were carried out inside the laboratory.

In practice, a 20% reduction is necessary for bridges built outside. Dead load deflections were ignored as they had already taken place before measurements commenced.

From these results, it is concluded that elastic step-wise linear analysis gives a very good estimate of the actual deflections and stresses in a timber stress laminated arch. It is also concluded that, as little as 30% of the full lateral tension, is sufficient to produce friction for load transfer. This was found to be the case in a similar set of tests in Brazil [80] where a set of trusses were stress laminated to form an orthotropic plate for bridge construction. Further work on stress laminated trusses would be a valuable extension of this work.

6.11 Dynamic Load Tests of Bridges in the Field

The parameters for dynamic effects were described in Section 5.5 of Chapter 5. The instrumentation was described in Section 6.2.1 of this chapter. A full description of the three methods of testing is given in Section 6.9.8 of this chapter which dealt with the 20m test span. However a number of subsequent tests were carried out on other bridges in the field to ascertain confirmation of the results. These tests were carried out using the hand held RT 440 instrument shown in Figure 6.14 on bridges before and after the bitumen topping was added. The results of these tests are shown at the end of Chapter 7.

It is a first mode natural frequency that could be emulated by live load and become a problem. The lighter the bridge, the greater natural frequency needs to be for safety. Evaluation of this relationship is not part of this research project but it depends on the number of vandals available and the number who can synchronise jumping. It is considered that a 100kN weight of bridge is safe with a natural frequency of 3.5Hz, whereas a 300kN may only need a natural frequency of 2Hz.

6.12 Materials Testing

In Section 5.21 the materials specification is described with reference to Appendix 1 and the visual grading technique described in Appendix 2. Section 6.3 described the further physical specification items concerning the timber used to build the arches. Generally grade C16 in accordance with BS 5268 [50] was used as that is what is readily available from home grown sources. However, it is well known that, because of the small quantities of timber required as stress graded in the UK there is little investment in grading machinery. This has resulted in much of the home produce all being classed as C16 when some of the timber is of a higher grade. For that reason and the fact that the visual grading method is imprecise it is quite likely that some of the timber used to build the arches had a different strength than assumed.

Modulus of Elasticity and density were used in the evaluation of the deflections in the analysis of the model arches used as comparisons with the experimental results. They were also used in calculations to build the semi empirical model in Chapter 8. Throughout all of the tests, a moisture content was measured using a meter which is a useful quick indicator but the results are not reliable. For these reasons a set of materials tests were carried out on timbers used to build the laboratory arches.

Eight samples were tested for bending from the timber used for the 2.1m span arches. Ten samples were tested for moisture content, bending and compression in three different directions on timbers used for the 6m spans. The results were averaged to give one set of values for each arch span. The tests were carried out in accordance with the requirements of BS 408 [81] and BS 373 [82] and the results are given in Section 7.8 of Chapter 7. Some extra details of the tests, preparation of samples and apparatus are given in Appendix 3.

The timber for the first 6m span arch and the 2.1m span laboratory arches was of poor quality. The 20m span test bridge used C16 visually graded Sitka Spruce and the laboratory 6m spans used good quality Sitka Spruce. The timbers used for permanent arch bridges were also visually stress graded but the species were Scots Pine or Douglas Fir chosen for durability and treatability. Good treatment of Sitka Spruce is still being researched. The materials testing will give an indication of the percentage difference that may exist between the actual strengths and the visually assessed strengths.

CHAPTER 7

7 DATA FROM EXPERIMENTAL WORK AND ANALYSIS

This chapter contains the results for all of the tested bridges to show load, deflection and stiffness relationships. It shows correlations with analyses and treatment of the results to derive relationships between independent variables. The results of the dynamic tests are also listed.

7.1 Data Management and Collection

Outputs using transducers and load cells were fed into Microsoft Excel to build spreadsheets, plots and charts to display and analyse data. The remote measurements of deflection taken by the 'SOKIA Total Station' were treated manually. The dynamic test results were recorded electronically on the 'RT 440' and converted for use in Excel.

7.2 6m Span Trial Arch

The results shown in Figure 7.1 are the load deflection plots for various loadings and un-loadings, at chosen positions across the bridge. A linear regression line, for the loaded actions only, is superimposed (heavy black dotted line) to represent the minimum stiffness at the position on the arch showing maximum deflection (the centre). Figure 7.2 shows that the deflection is symmetric for four point loading and exaggerated by a factor of four to make the shape clear.

The set of plots shown are for the maximum loading up to 50kN.

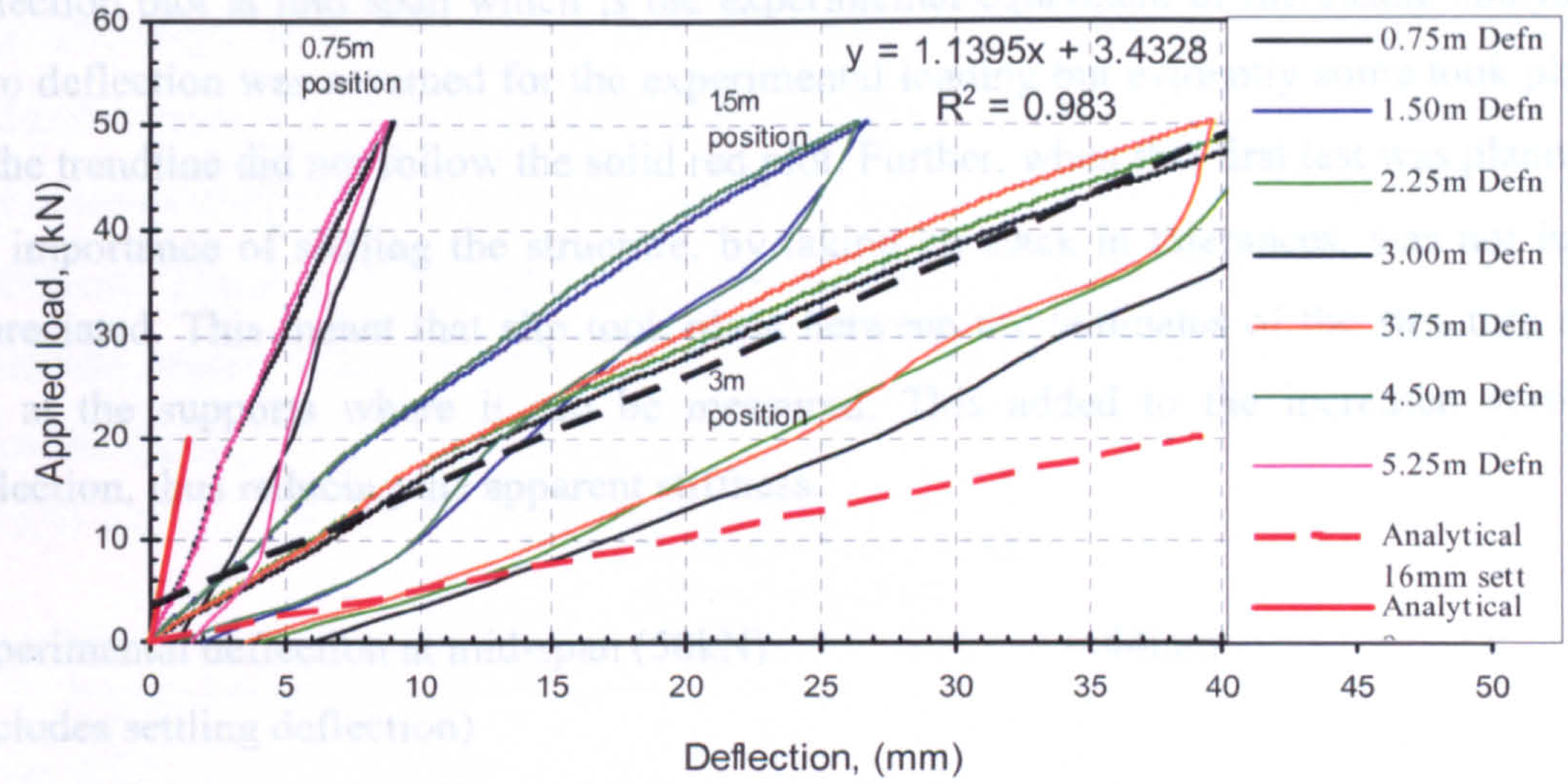


Fig 7.1 – 6m span - 50kN - four point loading

Load deflection curves with no support settlement

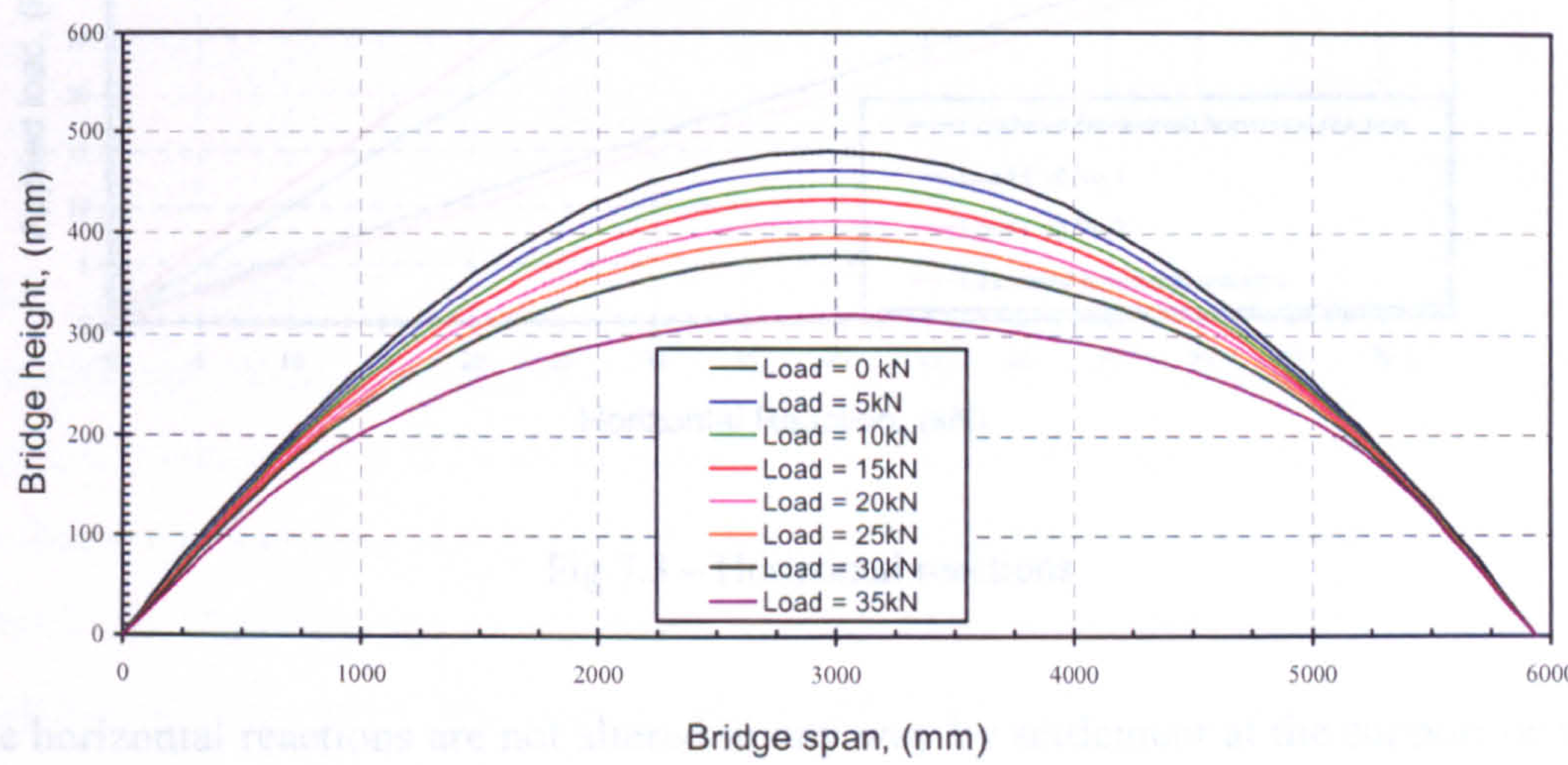


Fig 7.2 – Arch shape with increasing loads

Figure 7.1 shows two plots, in a heavy red line, for deflection at the midspan for four point loading from an elastic analysis. One has allowed no settlement and the other (dotted line) 16mm of settlement. This gives a very good indication of the sensitivity of the vertical deflection to settlement. The heavy black dotted line is a trendline of the deflection plot at mid span which is the experimental equivalent of the elastic analyses. Zero deflection was assumed for the experimental loading but evidently some took place as the trendline did not follow the solid red plot. Further, when this first test was planned, the importance of settling the structure, by taking up slack in tolerances, was not fully appreciated. This meant that slip took place between the laminates of the structure and not at the supports where it can be measured. This added to the increased vertical deflection, thus reducing the apparent stiffness.

Experimental deflection at mid-span (50kN) (includes settling deflection)		= 44mm
Calculated deflection at mid-span (50kN)	δ	= 4mm
Experimental Test:	K	= 1.1395kN/mm
Calculated:	K	= 13.89kN/mm

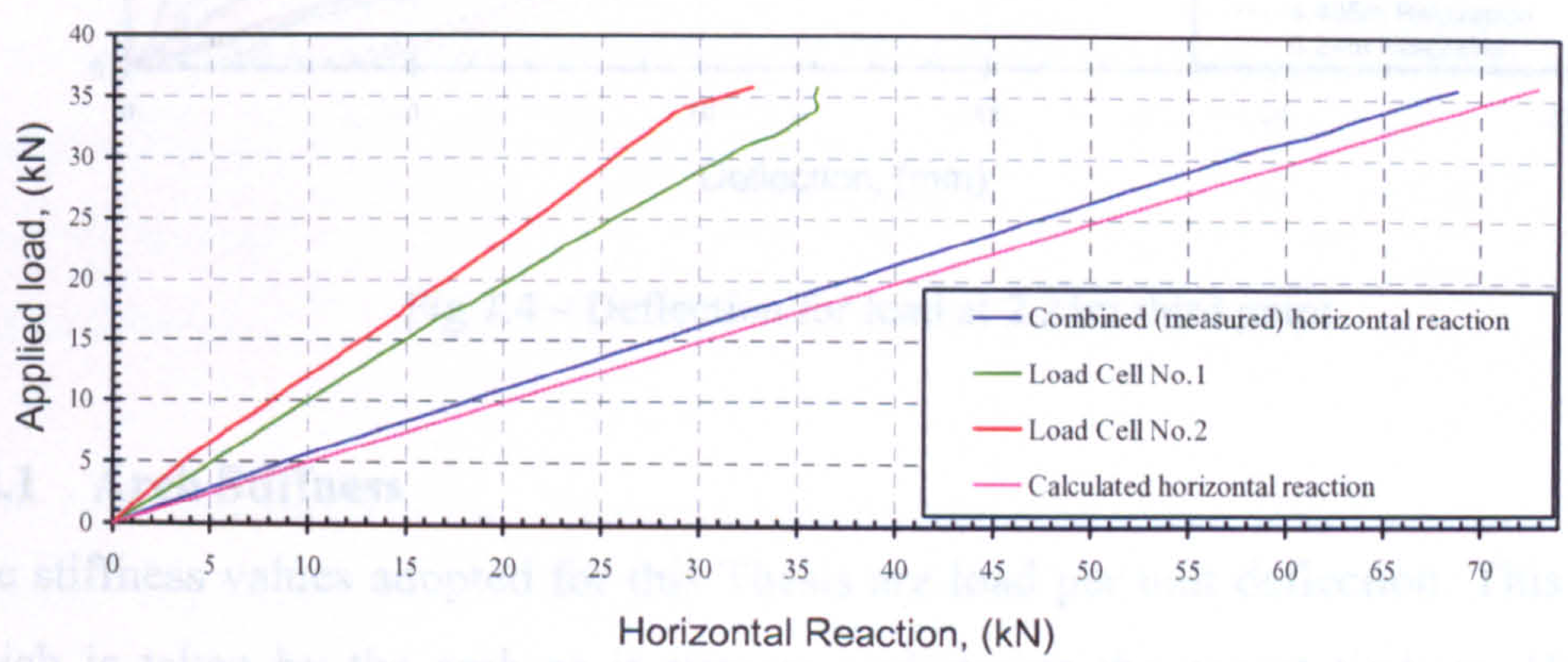


Fig 7.3 – Horizontal reactions

The horizontal reactions are not altered in any way by settlement at the support or within the structure. A good correlation between the measured and linear elastic analysis was observed in Figure 7.3.

The results from these tests showed the value of the structure by demonstrating very large load capacity, using low quality timber. However, the results do not give good indications of stiffness, which is one reason that a series of 6m spans were tested later.

Below Figure 7.4 is a load-deflection relationship for a line load at the third point of the arch. The experimental deflection is a little over 20mm, with no settlement assumed. Under the same conditions, the calculated deflection is 7mm which again shows there was internal settlement of the arch cause by slip of the laminates and some lateral settlement. These shallow arches are very sensitive to small lateral movements. The geometric secular shape of rise to span of 1 to 12 means that vertical deflection is 2.4 times the support yield.

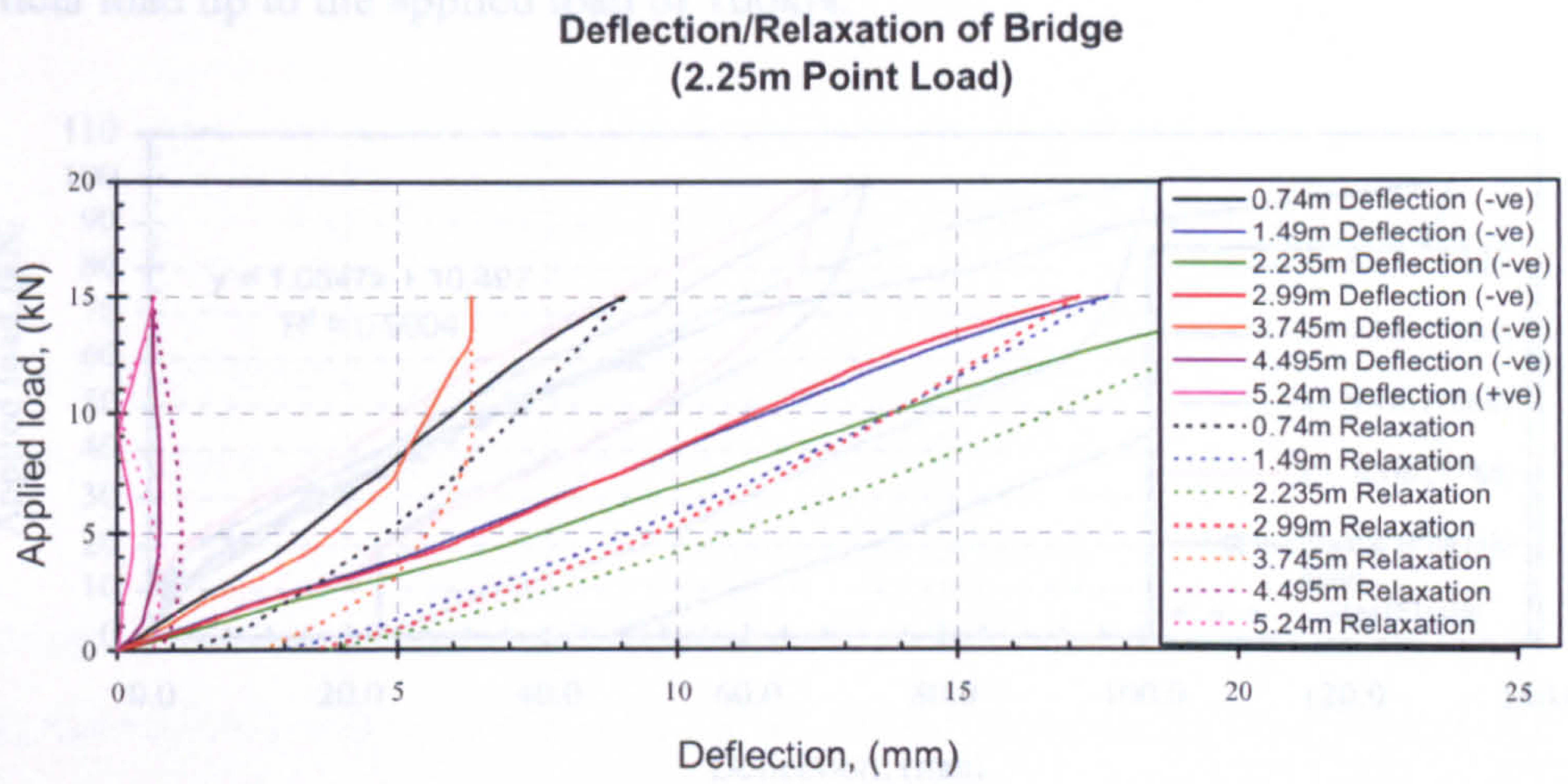


Fig 7.4 – Deflection for load at 2.25m third point

7.2.1 Arch Stiffness

The stiffness values adopted for this Thesis are load per unit deflection. This is the load which is taken by the arch as it stresses and strains the parent timbers. However the laminates and supports of a stress laminated arch slip and settle, causing movements and deflections which do not create stress, so they are not indicative of stiffness. **For the purpose of this study stiffness is, therefore, the load divided by the maximum deflection within the elastic limits of the structure**. Such an approach gives **stiffness** as a common base line for comparison of all arches. All internal settlements and support settlements are assumed to result in changes of shape, without inducing stress.

STIFFNESS = LOAD PER UNIT DEFLECTION (within elastic limits)

7.3 15m Span Permanent Arch

Figure 7.5 shows the load deflection behaviour for various loadings and un-loadings, at chosen positions across the bridge for uniform load in the middle third. It shows the minimum stiffness of the arch at mid-span as a trendline (heavy black dotted line) of the elastic zone. Figure 7.6 shows the support yield for applied vertical load. These settlement values are used in an elastic analysis to calculate equivalent analytical values for vertical deflection of the arch. Figure 7.7 shows the horizontal thrust for progressive vertical load up to the applied load of 100kN.

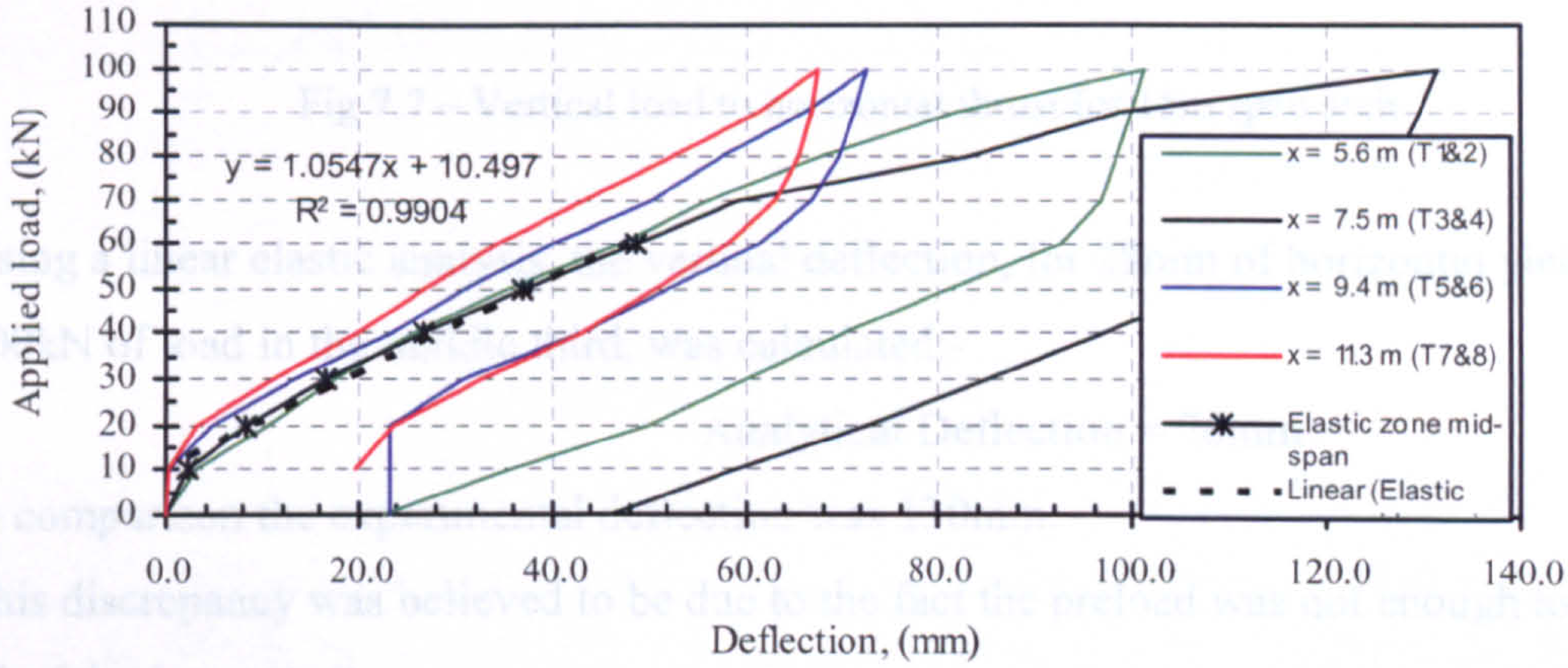


Fig 7.5 - Vertical deformation of 15m span arch

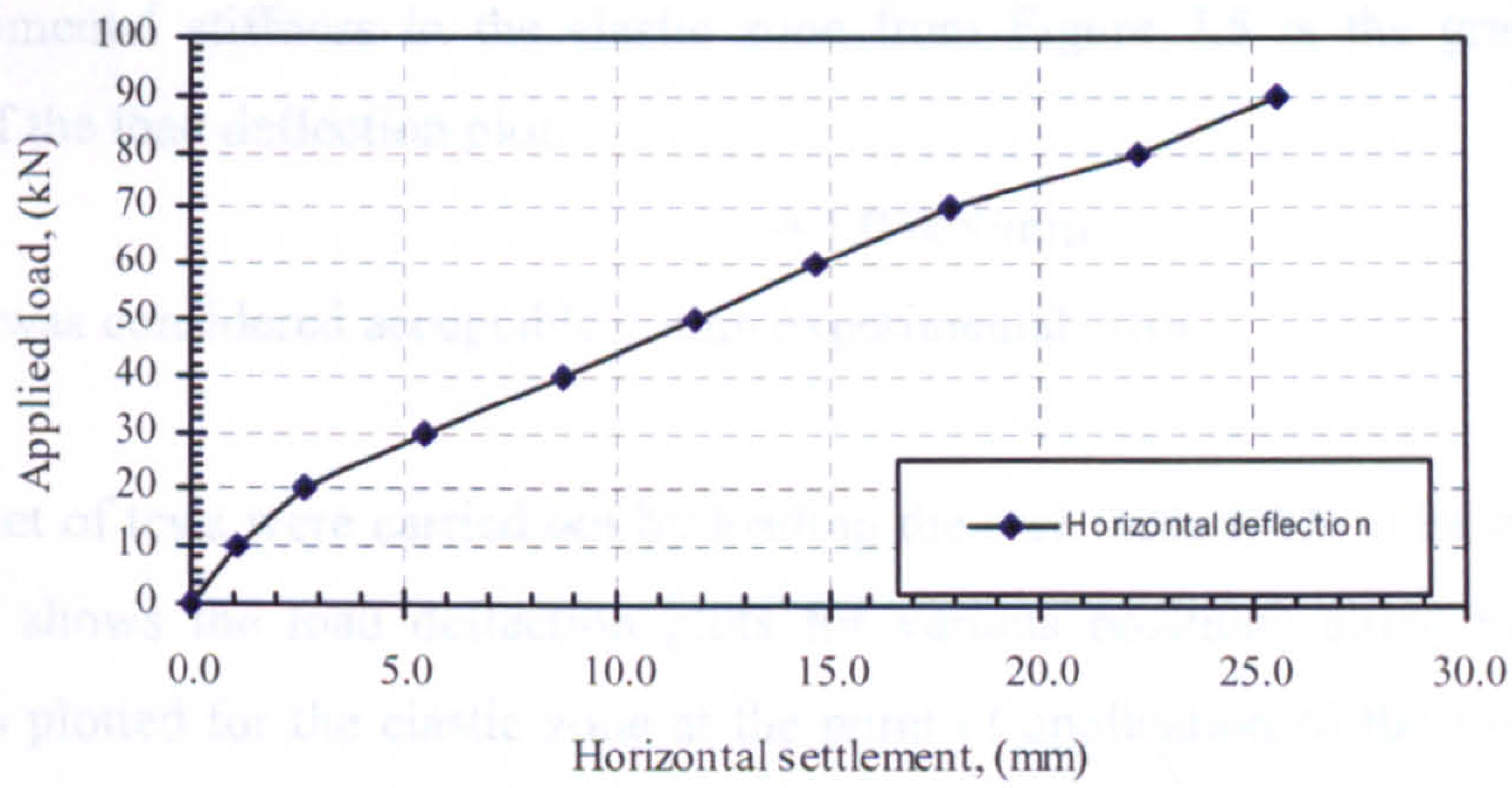


Fig 7.6 – Support yield for vertical load of 15m span arch

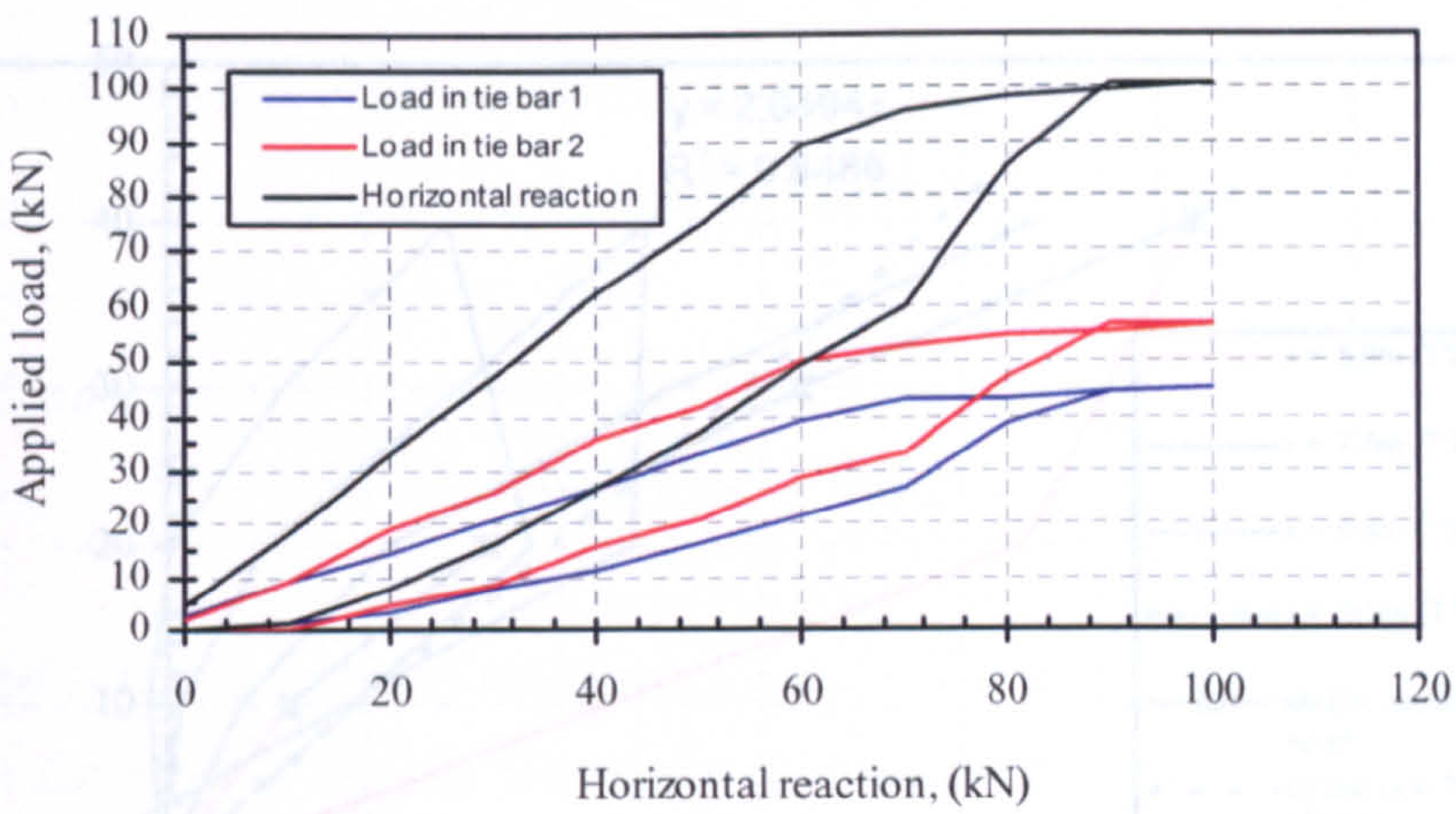


Fig 7.7 – Vertical load to horizontal thrust for 15m span arch

Using a linear elastic analysis, the vertical deflection, for 28mm of horizontal yield, and 100kN of load in the middle third, was calculated:-

Analytical Deflection = 76mm

In comparison the experimental deflection was 130mm.

This discrepancy was believed to be due to the fact the preload was not enough to remove all of the internal slip.

The analytical stiffness is:-

$$100/76 = 1.32\text{kN/mm}$$

The experimental stiffness in the elastic zone from Figure 7.5 is the gradient of the trendline of the load deflection plot:-

$$= 1.05\text{kN/mm}$$

Again this was considered acceptable within experimental error.

A second set of tests were carried out by loading the arch with 40kN at its quarter point. Figure 7.8 shows the load deflection plots for various positions along the arch. The trendline is plotted for the elastic zone at the point of application of the load where the maximum deflection took place. Figure 7.9 shows the profile of the arch under progressive quarter point load.

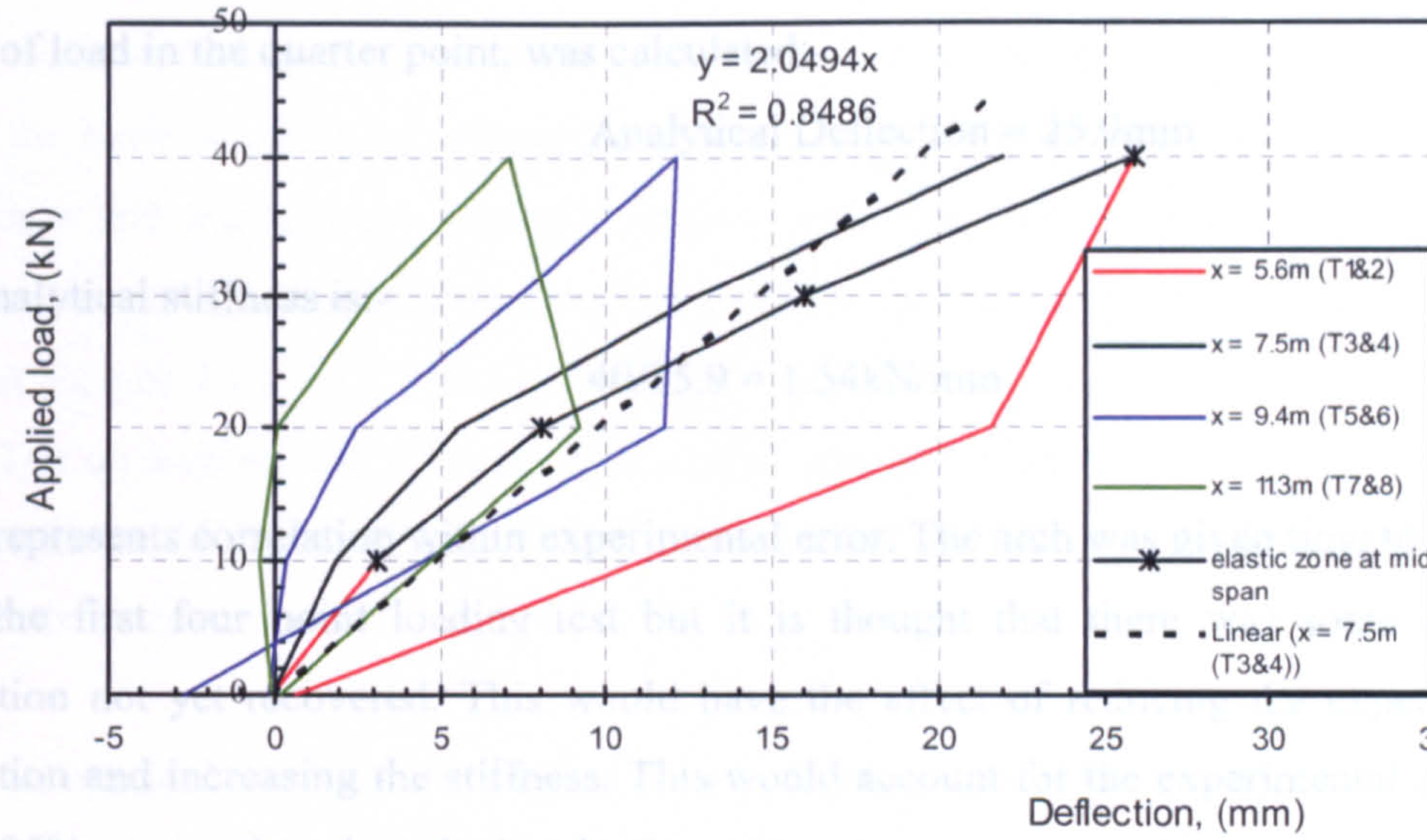


Fig 7.8 – Load deflection plots for quarter point loading on 15m span arch

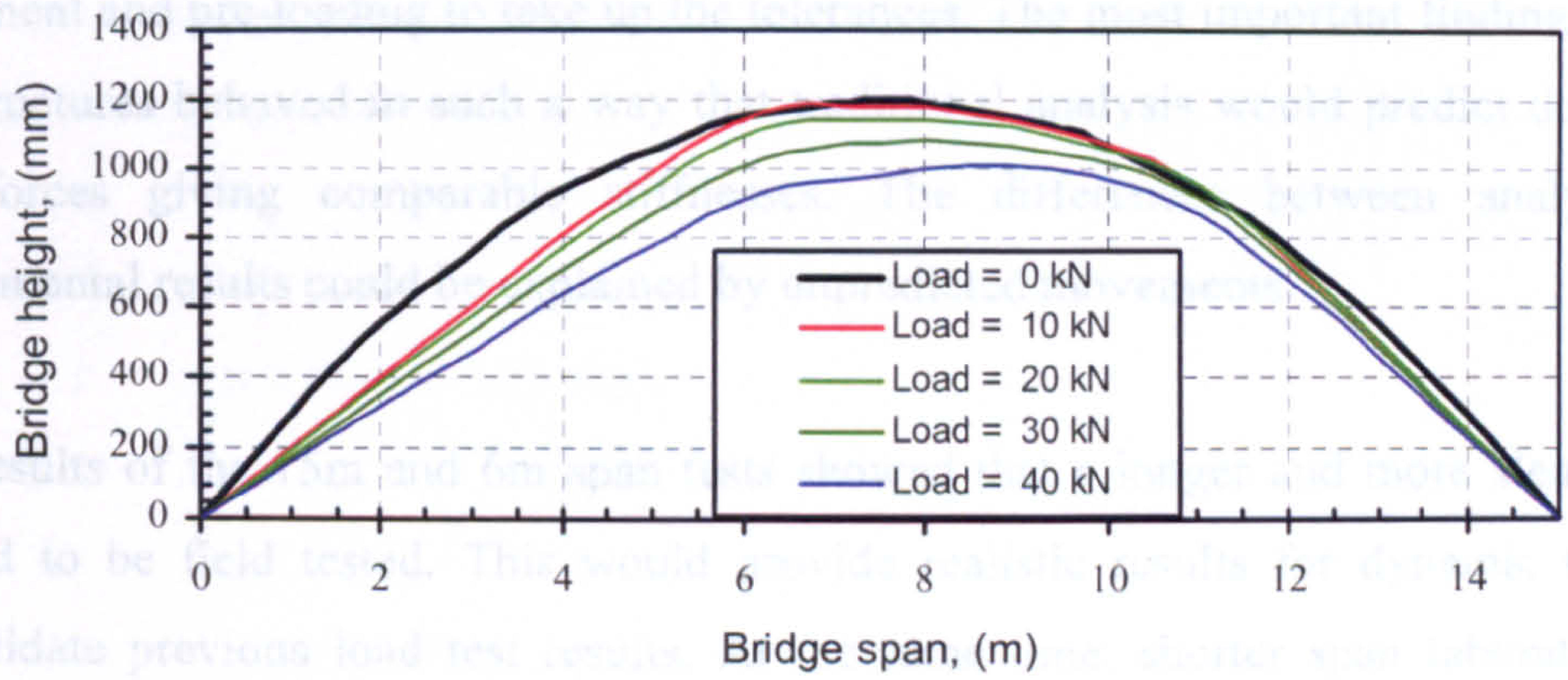


Fig 7.9 – Exaggerated deflected profile – quarter point loading 15m span arch

The stiffness of the 15m span arch at the quarter point for load at quarter point is again the gradient of the trendline, shown as a heavy black dotted line, in Figure 7.8:-

Experimental Stiffness = 2.049kN/mm

The support yield for 40kN of vertical load can be taken from Figure 7.6 as 8mm.

Using a linear elastic analysis, the vertical deflection, for 8mm of horizontal yield, and 40kN of load in the quarter point, was calculated:-

Analytical Deflection = 25.9mm

The analytical stiffness is:-

40/25.9 = 1.54kN/mm

This represents correlation within experimental error. The arch was given time to recover after the first four point loading test but it is thought that there was some residual deflection not yet recovered. This would have the effect of reducing the experimental deflection and increasing the stiffness. This would account for the experimental stiffness being 25% greater than the calculated value.

These tests, on the 15m span bridge, showed that the results from the half scale 6m span were relevant at full scale. It demonstrated the importance of minimising the lateral settlement and pre-loading to take up the tolerances. The most important finding was that the structures behaved in such a way that traditional analysis would predict deflections and forces giving comparable stiffnesses. The differences between analysis and experimental results could be explained by unpredicted movements.

The results of the 15m and 6m span tests showed that a longer and more slender span needed to be field tested. This would provide realistic results for dynamic tests and consolidate previous load test results. At the same time, shorter span laboratory tests would be set up to confirm correlation between experimental loading and analysis. The factors which created experimental error in the initial load tests would be considered carefully to ensure that they would not cause similar errors in later tests.

The load/deflection stiffness values from the above tests were considered to be a very useful comparative parameter. It was therefore decided to use this as the criteria to compare future results and, eventually, as the basis for comparing all variables when building a semi empirical model in Chapter 8.

7.4 2.1m Span Laboratory Scale Arches.

This series of secular 2.1m test spans, together with a subsequent series of 6m spans, forms the basis of controlled experimental load tests to determine how stiffness relates to arch rises and lateral transverse stiffnesses. Loads are plotted against deflections for different lateral tensions, different loading positions and for arches of different rises. The plots in Figures 7.10, 7.11, 7.12 and 7.13 show the loading curve to the maximum applied load. The un-loading which was also recorded, demonstrated full but delayed recovery. The gradients of the trendlines shown, are measures of stiffness and these are plotted against the lateral tensions.

The experimental results are compared to analytical for the 335mm rise span. All arches were 210mm wide with laminates having three holes, so the effective width is ⅔ of 210mm and the depth of the arch ring was 70mm.

The arches were loaded to 10kN at the lower lateral tensions and to 20kN at the highest. The arches were then loaded to failure with a progressive line load at the quarter point. The key to the graphs is as follows:

A0	Arch Rise	zero	
A1	Arch Rise	335mm	
A2	Arch Rise	447mm	
A3	Arch Rise	580mm	
CE	4 point loading i.e. simulated UDL		
QU	Quarter Point Load		
		Arches	Flat Deck
T1	Tension	Finger tight	Finger tight
T2	Tension	3.71kN	5.75kN
T3	Tension	8.21kN	11.49kN
T4	Tension	12.64kN	17.23kN

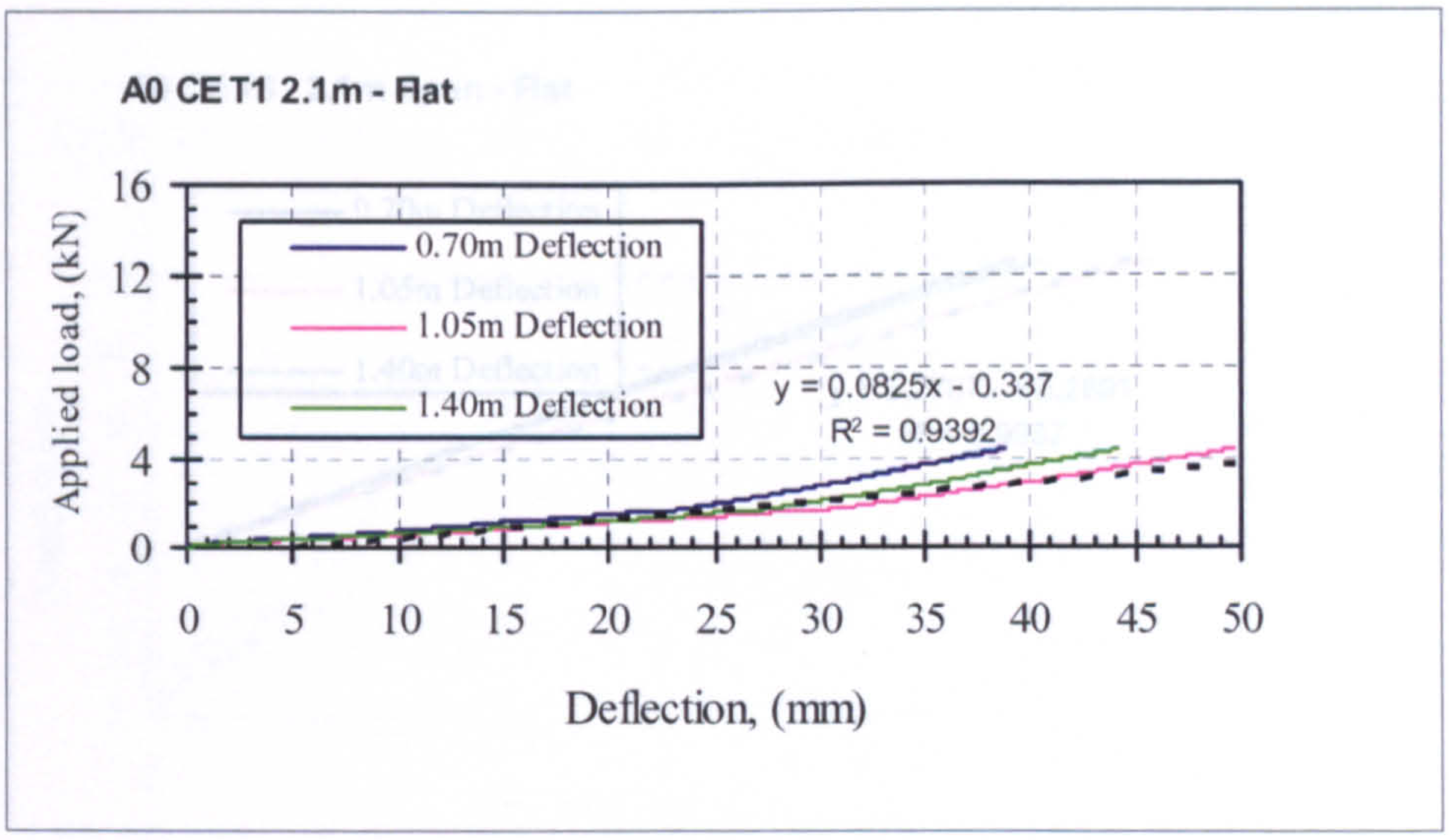


Fig 7.10a – Flat bridge with zero tension

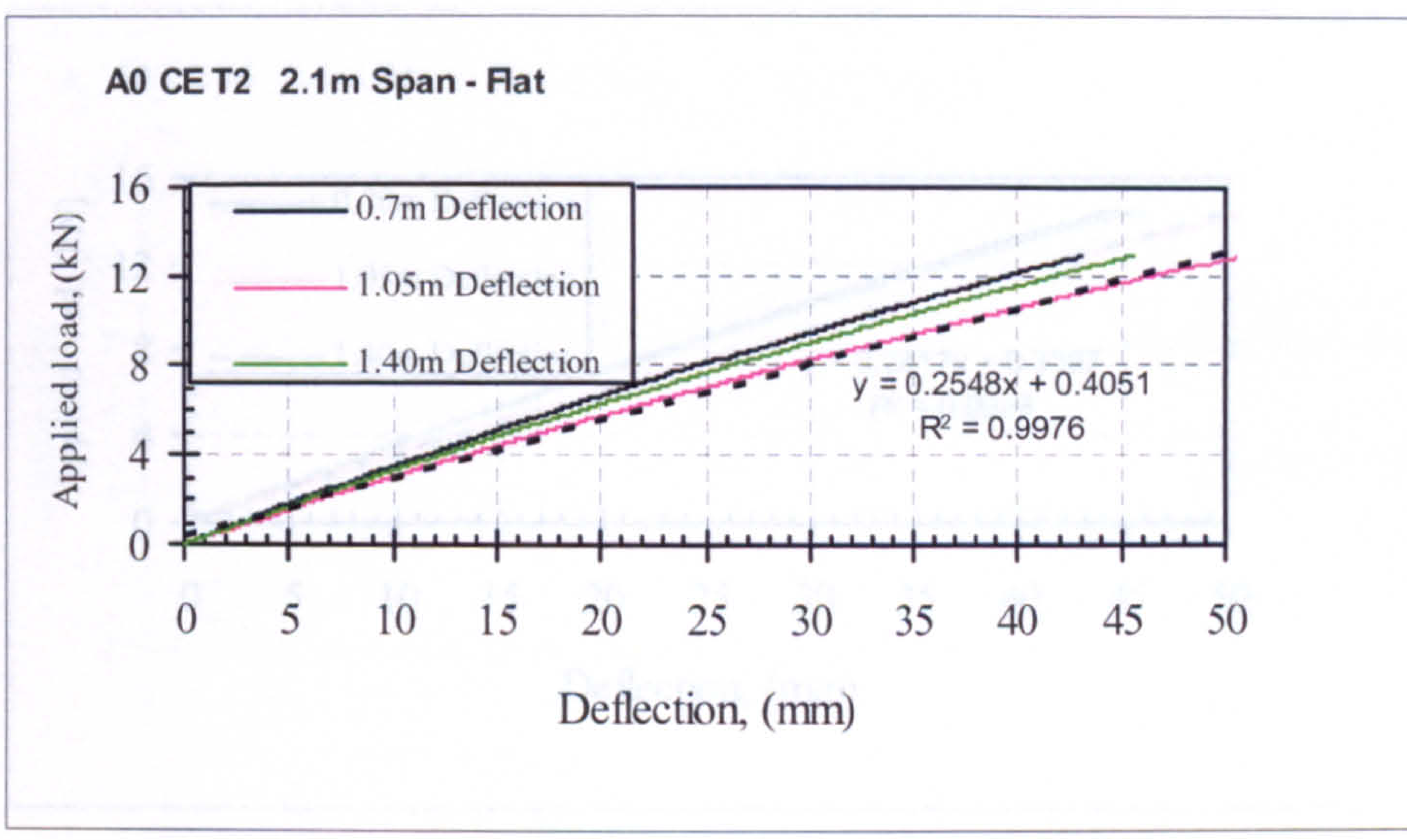


Fig 7.10b – Flat bridge with 5.76kN tension

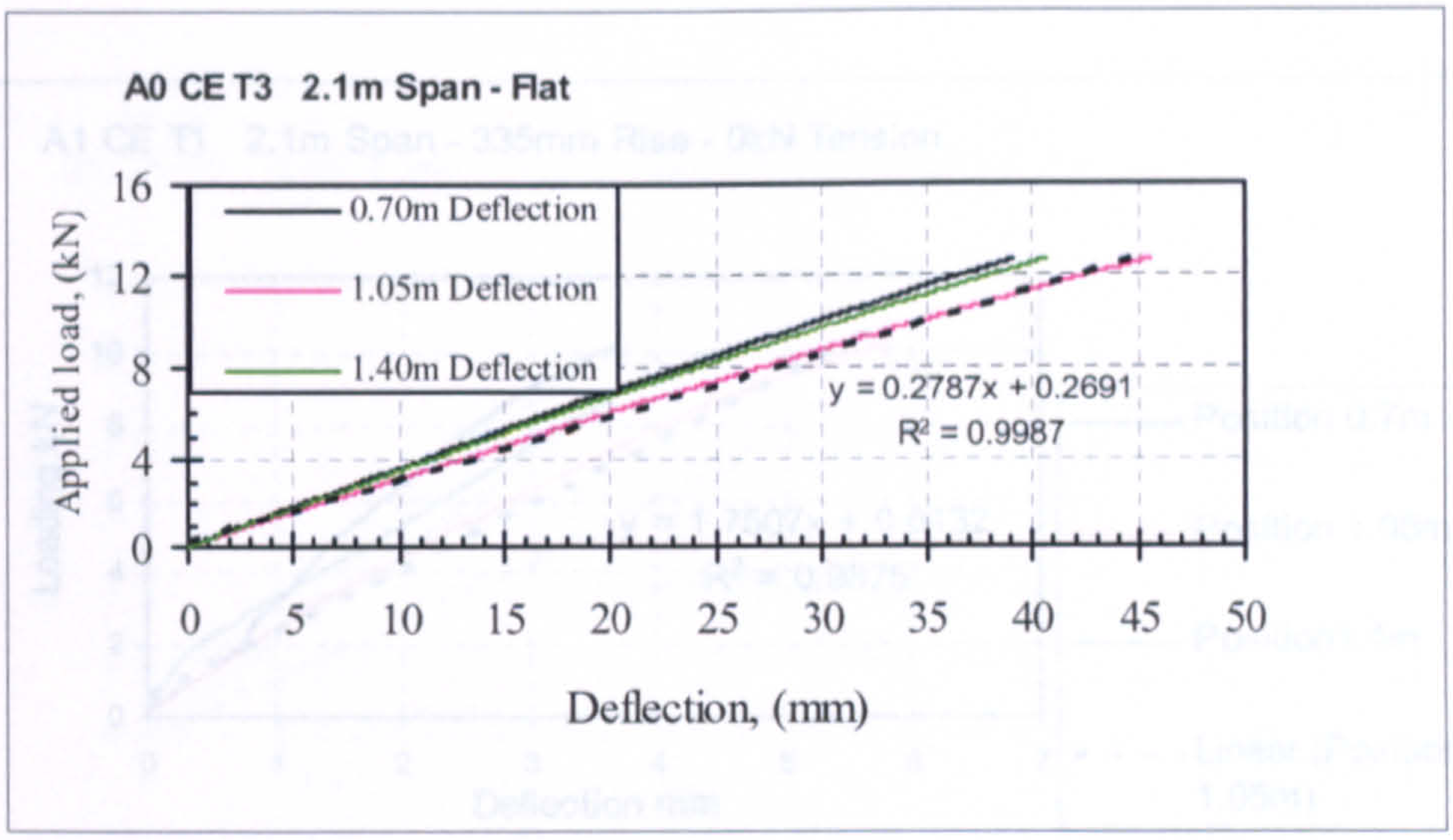


Fig 7.10c – Flat bridge with 11.49kN tension

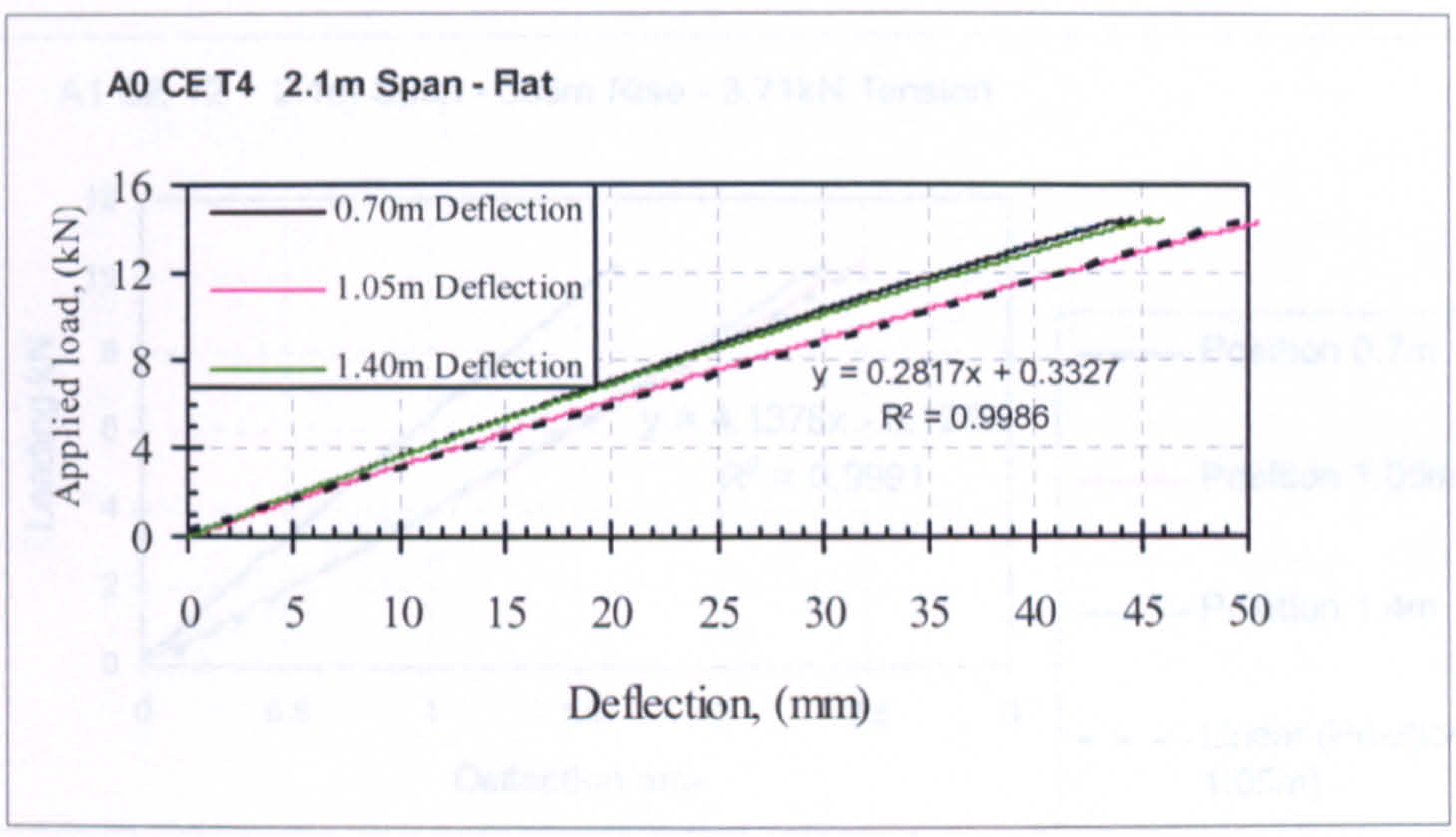


Fig 7.10d – Flat bridge with 17.23kN tension

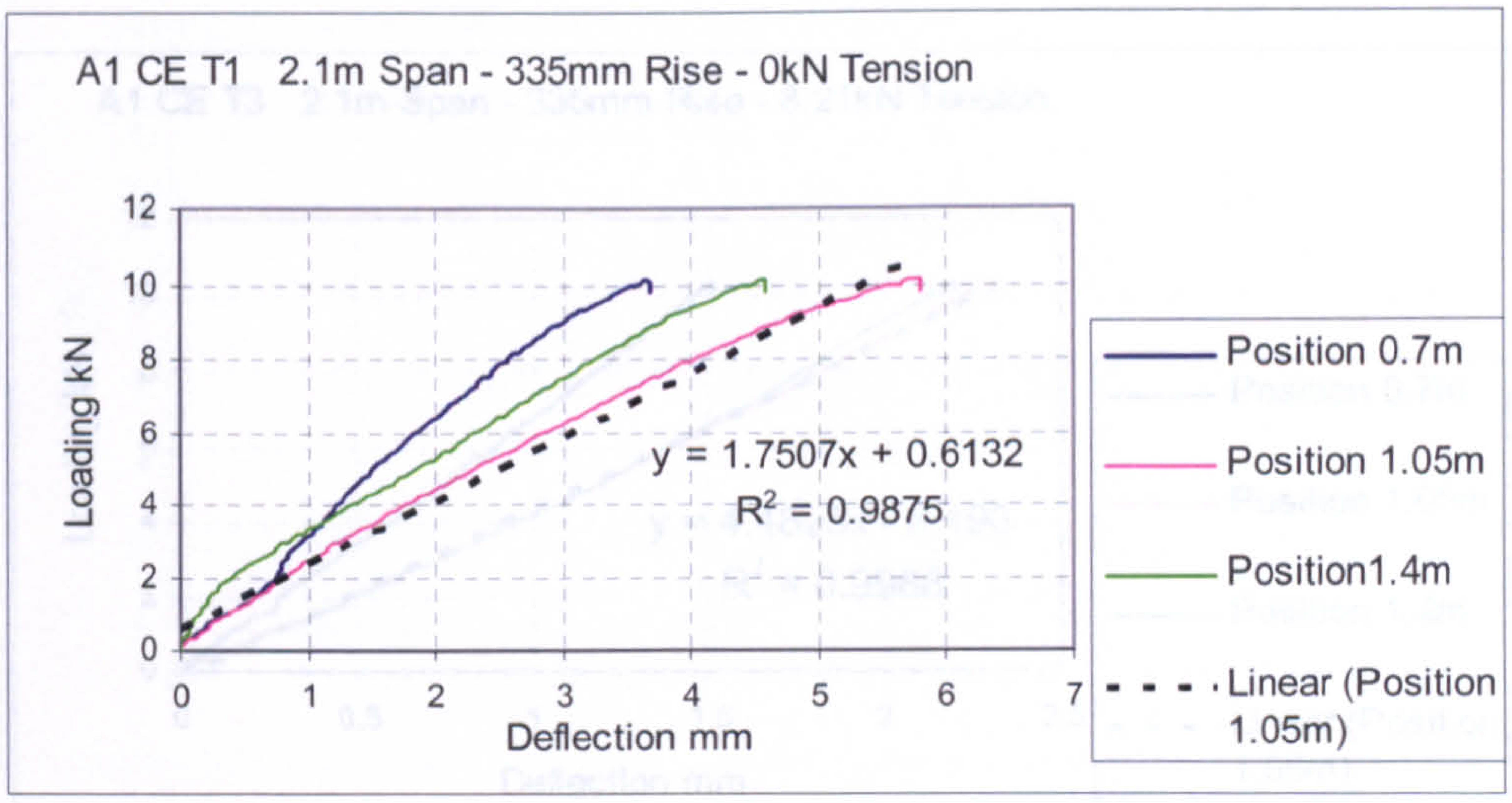


Fig 7.11a – 2.1m span – 335mm rise – zero tension

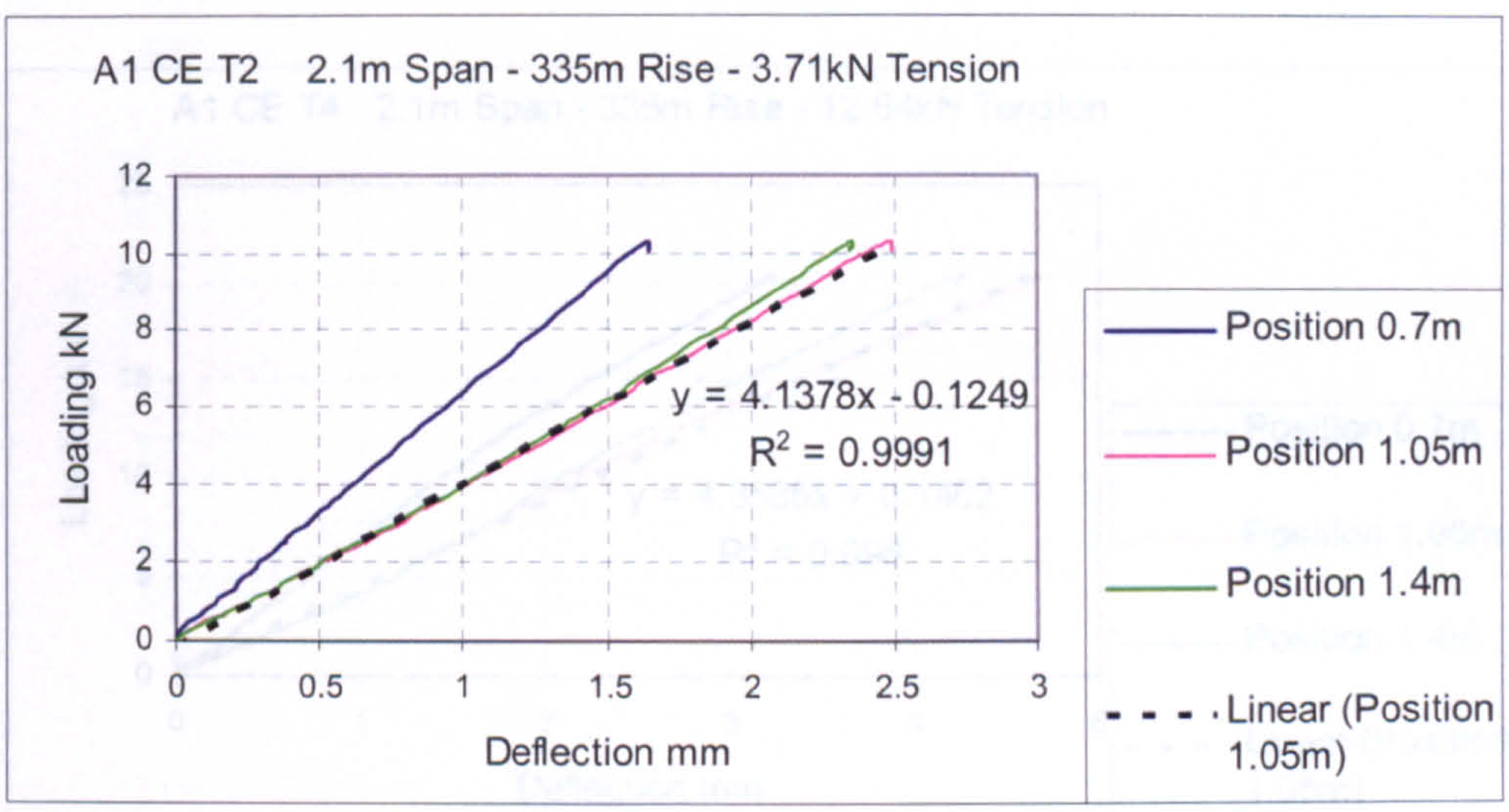


Fig 7.11b – 2.1m span – 335mm rise – 3.71kN tension

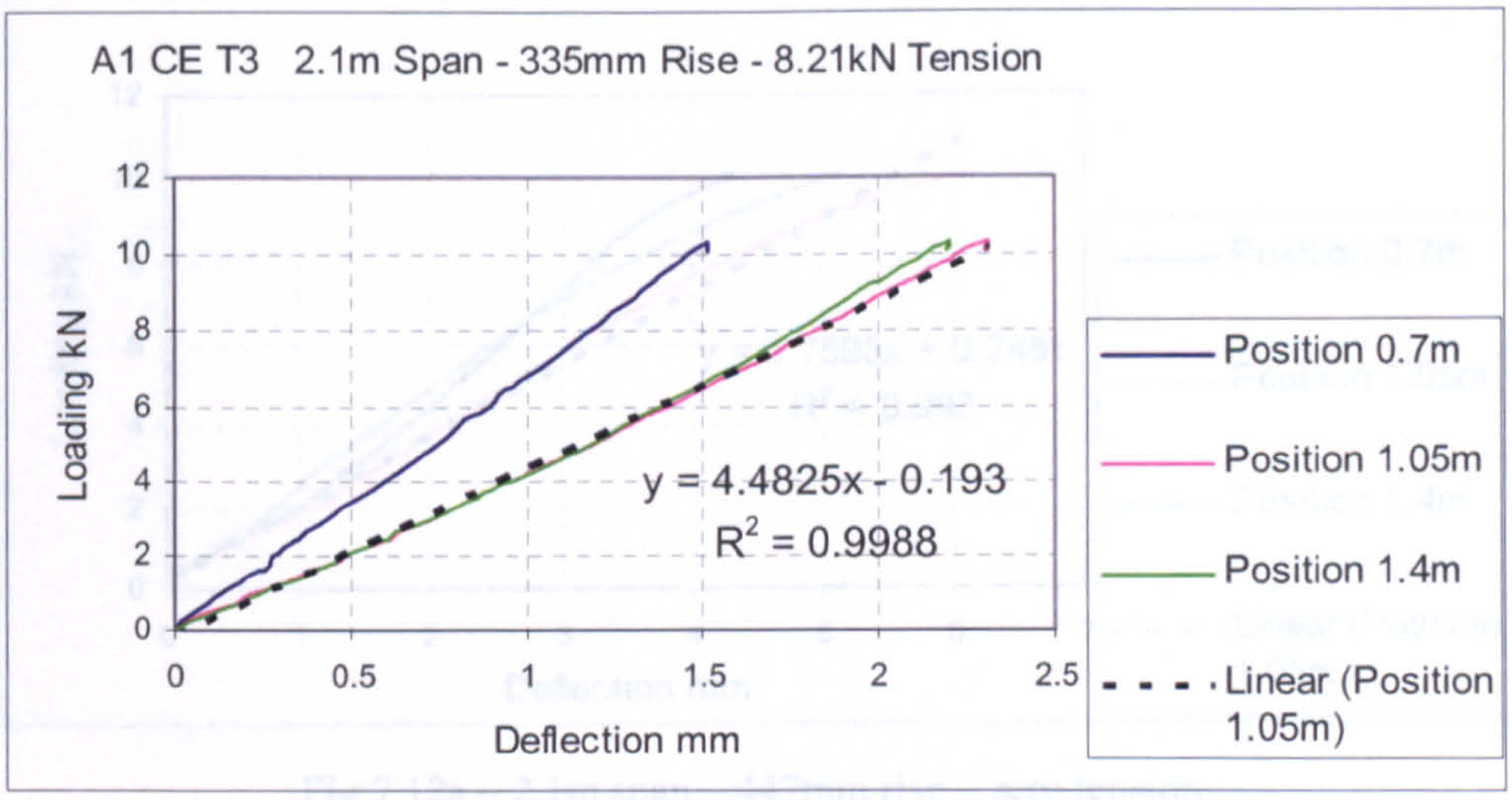


Fig 7.11c – 2.1m span – 335mm rise – 8.21kN tension

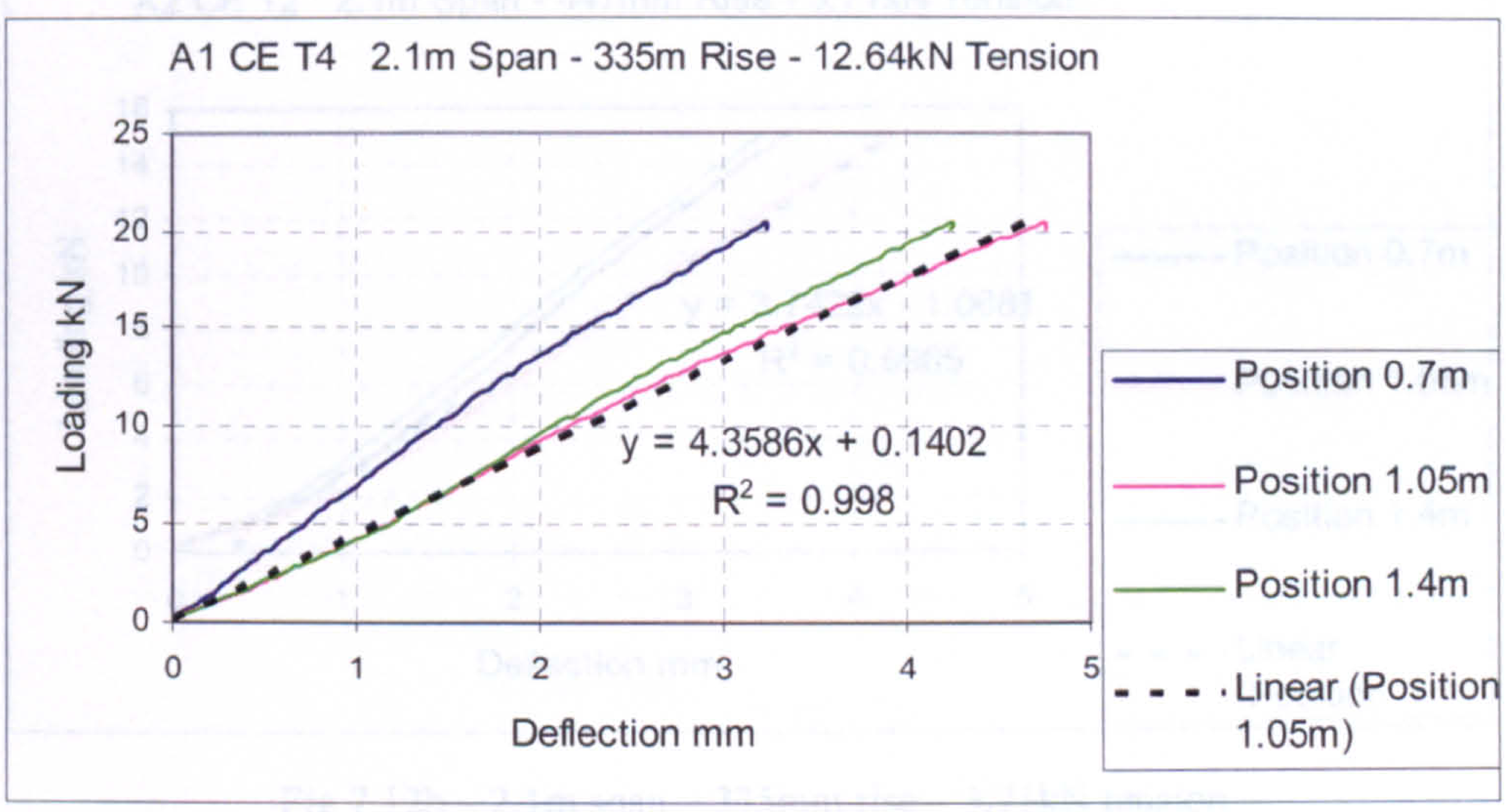


Fig 7.11d – 2.1m span – 335mm rise – 12.64kN tension

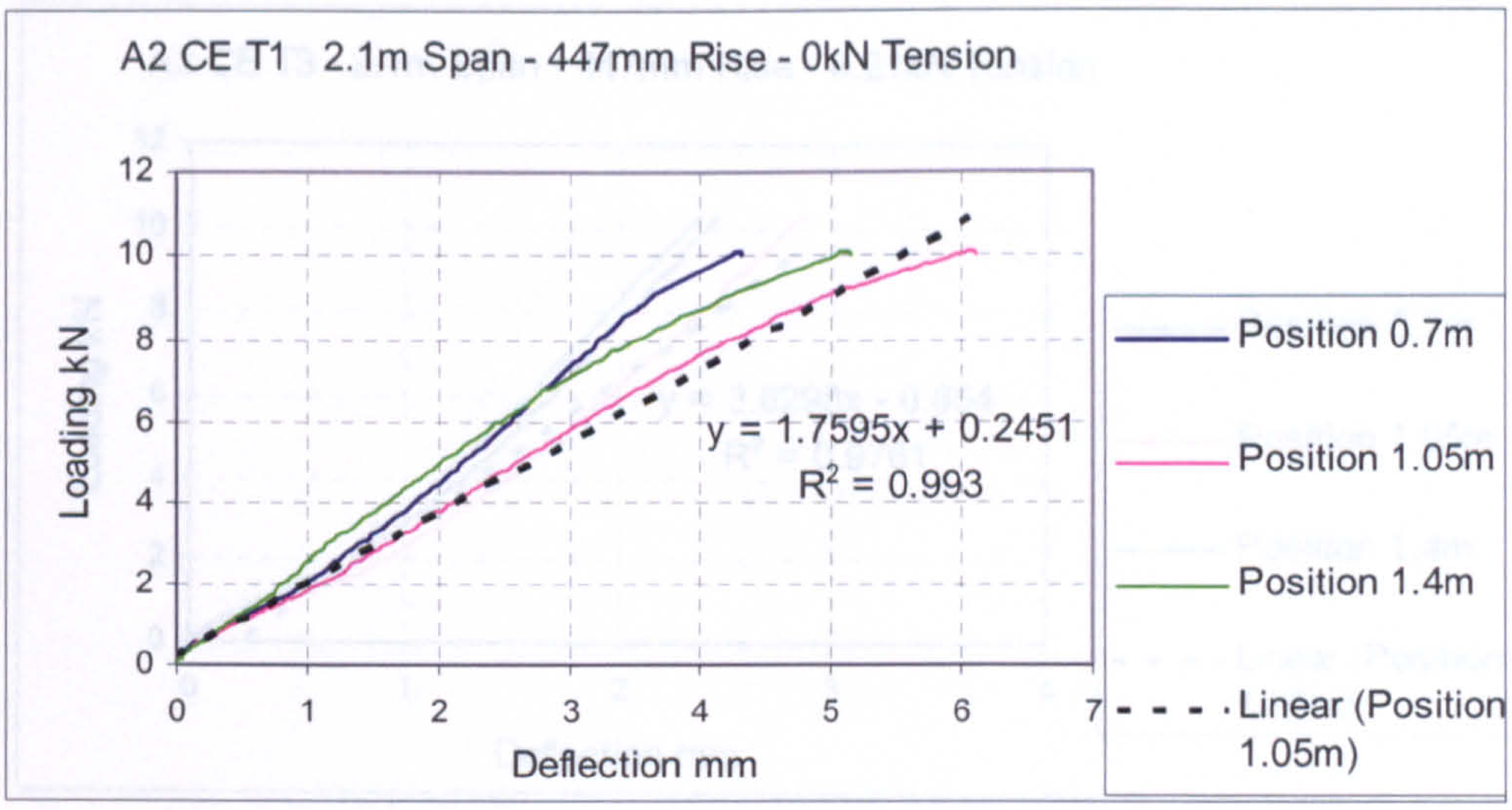


Fig 7.12a – 2.1m span – 447mm rise – zero tension

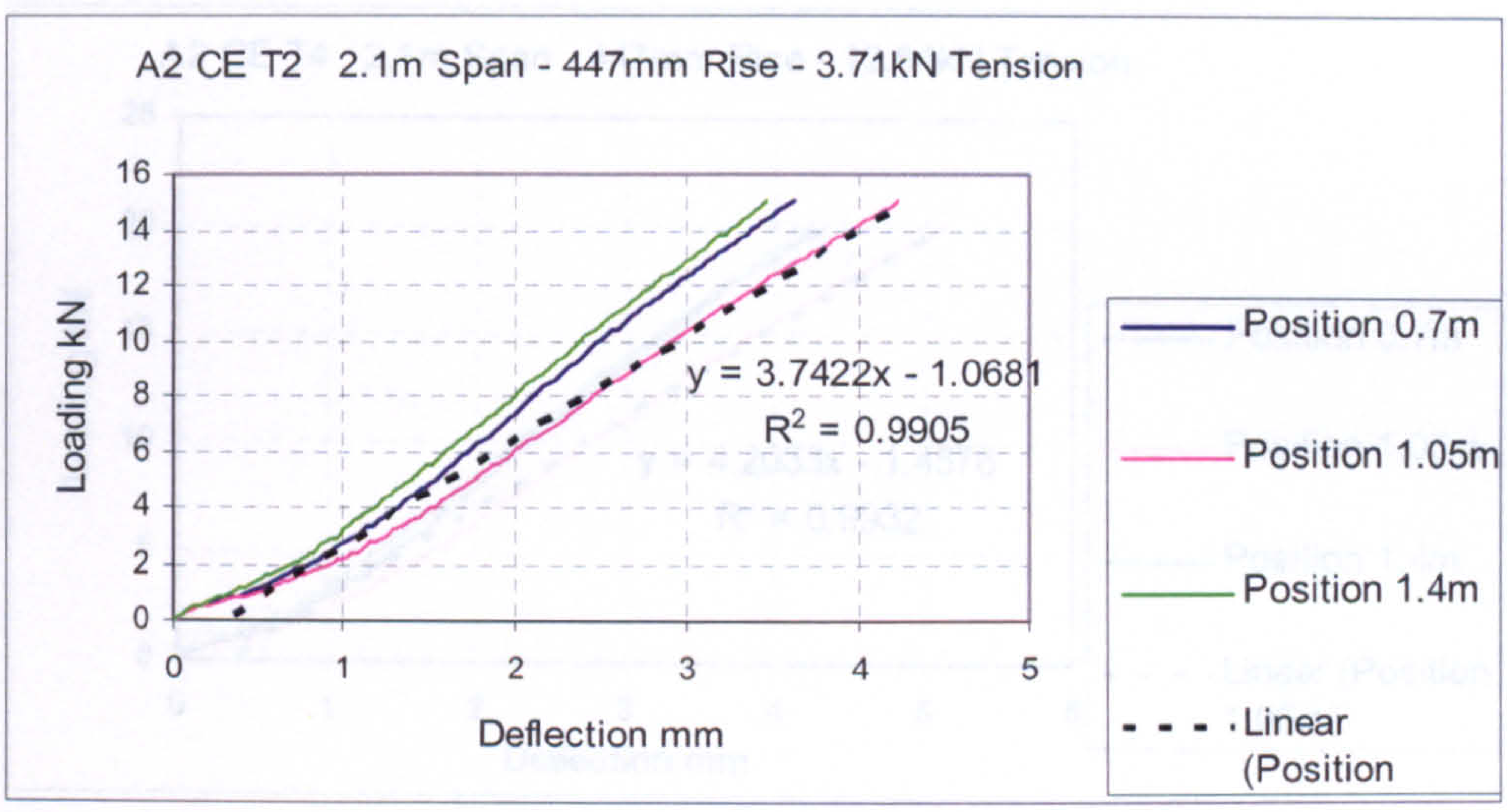


Fig 7.12b – 2.1m span – 335mm rise – 3.71kN tension

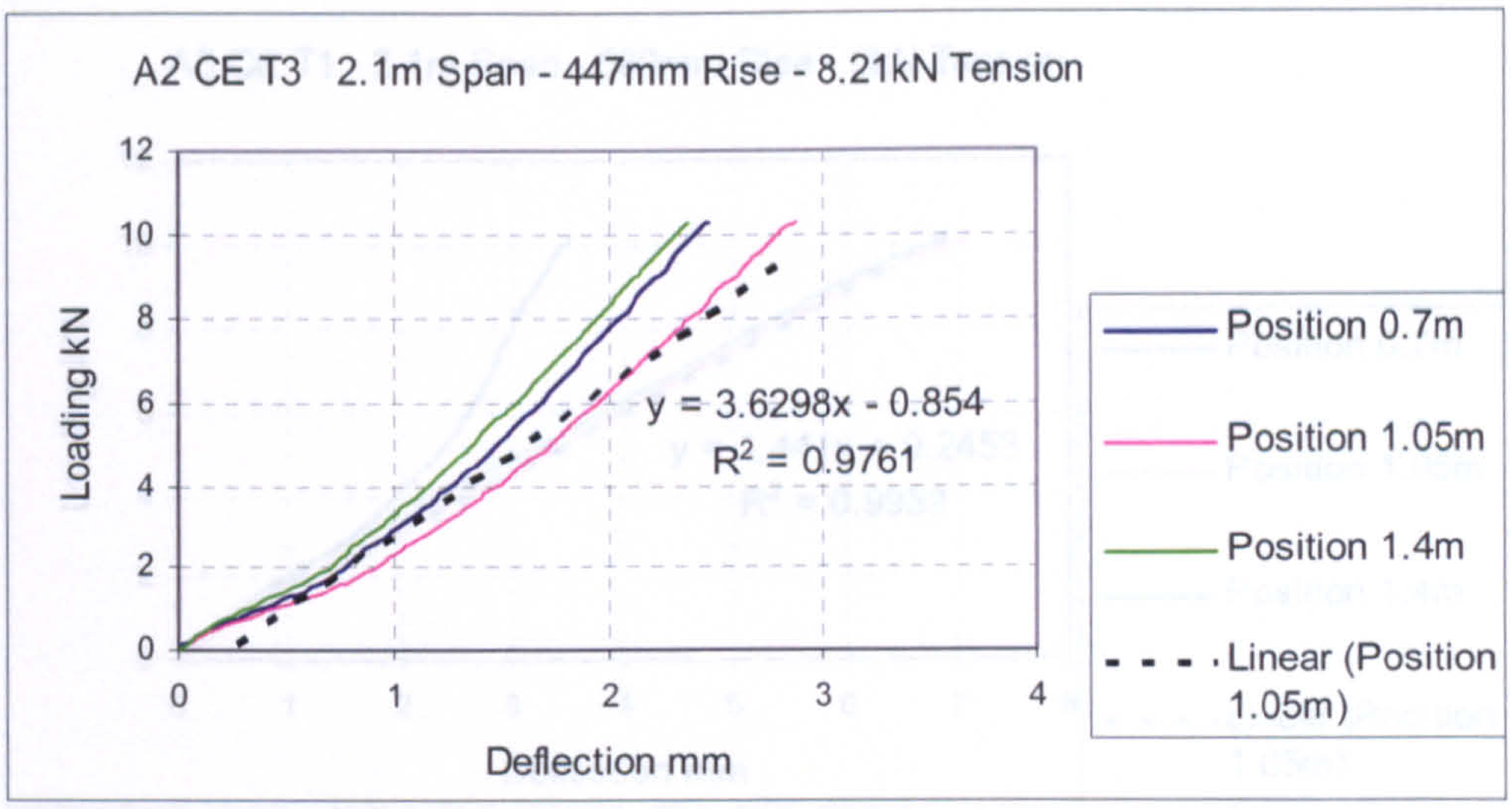


Fig 7.12c – 2.1m span – 447mm rise – 8.21kN tension

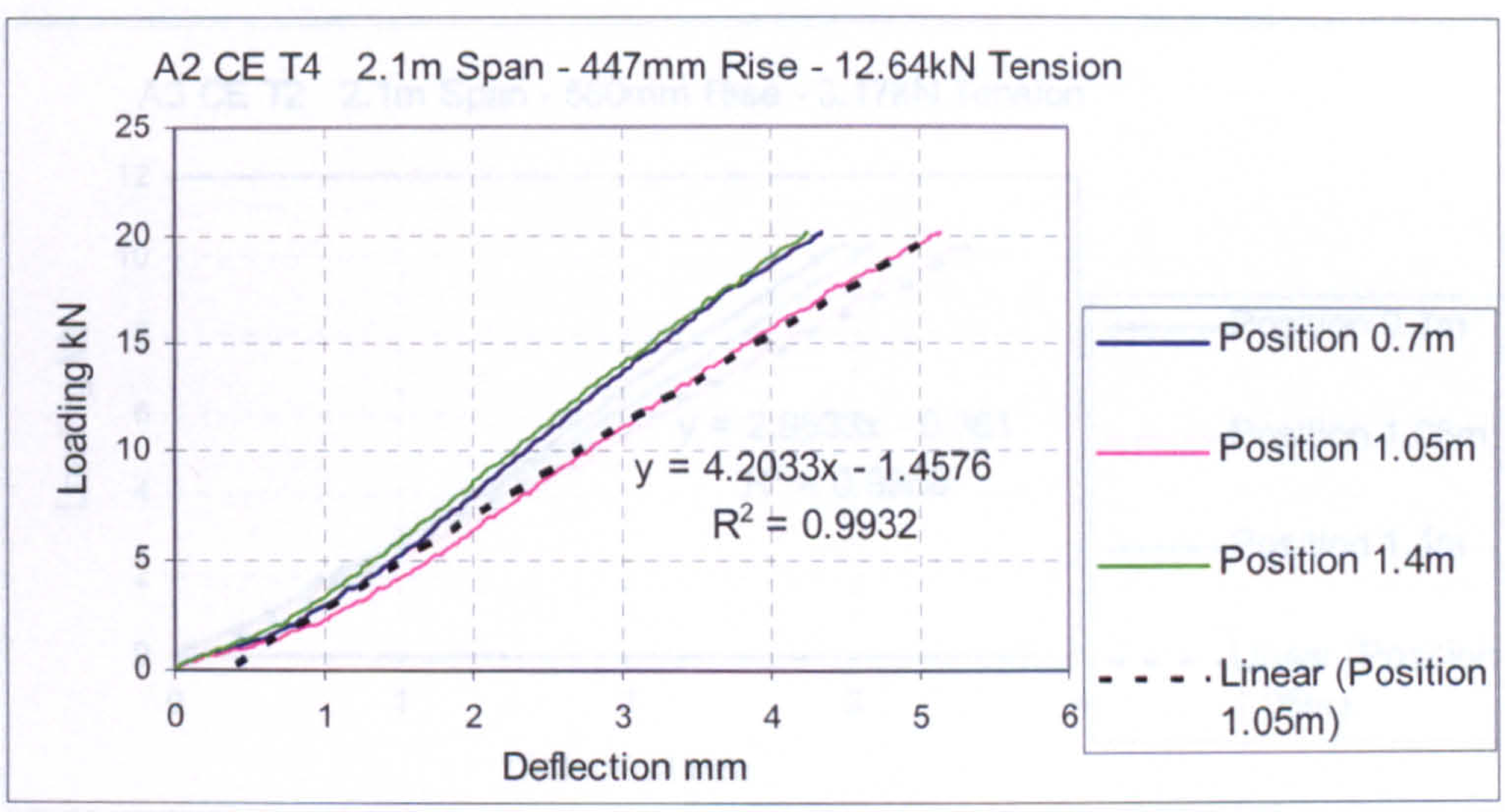


Fig 7.12d – 2.1m span – 447mm rise – 12.64kN tension

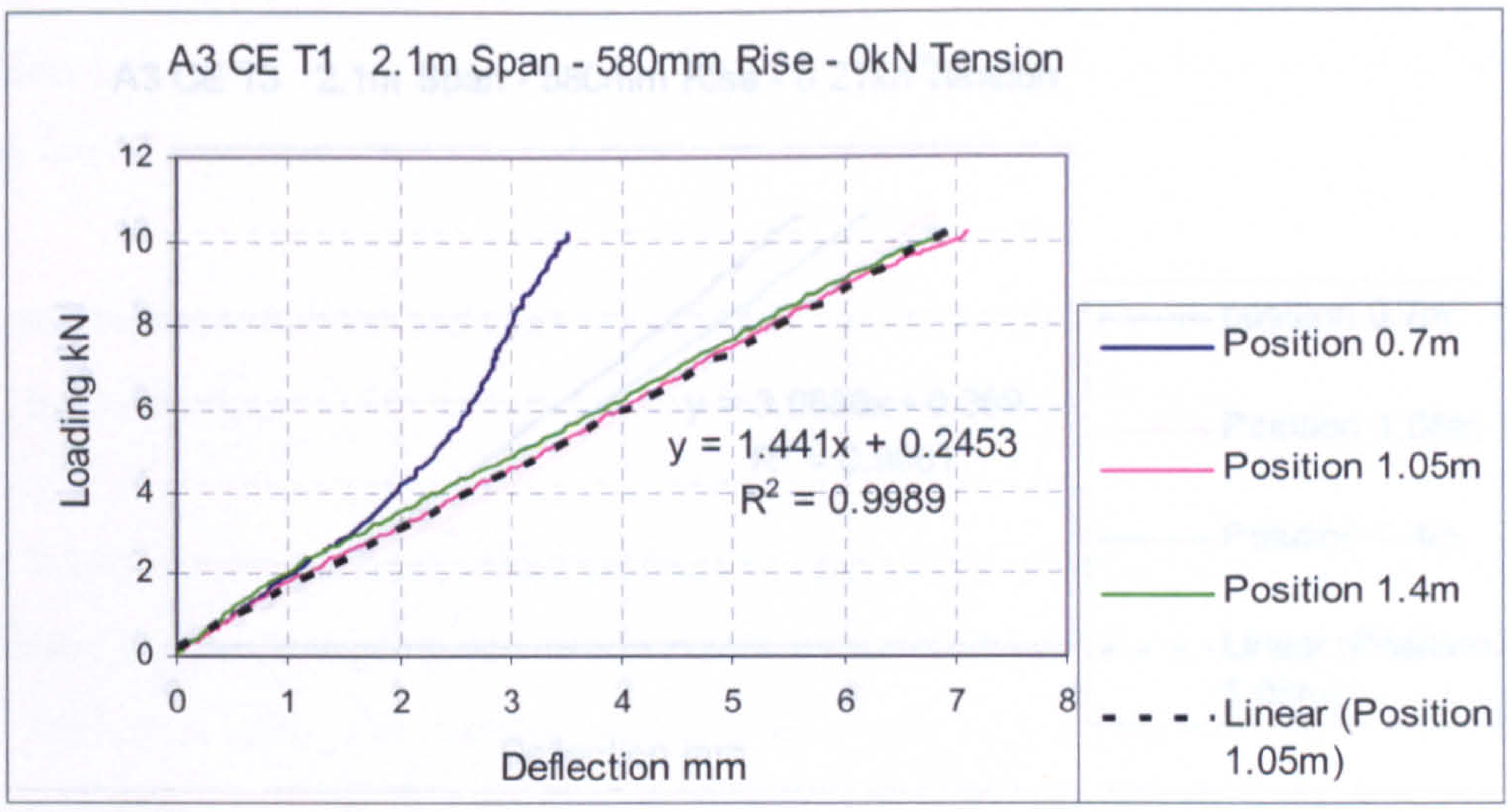


Fig 7.13a – 2.1m span – 580mm rise – zero tension

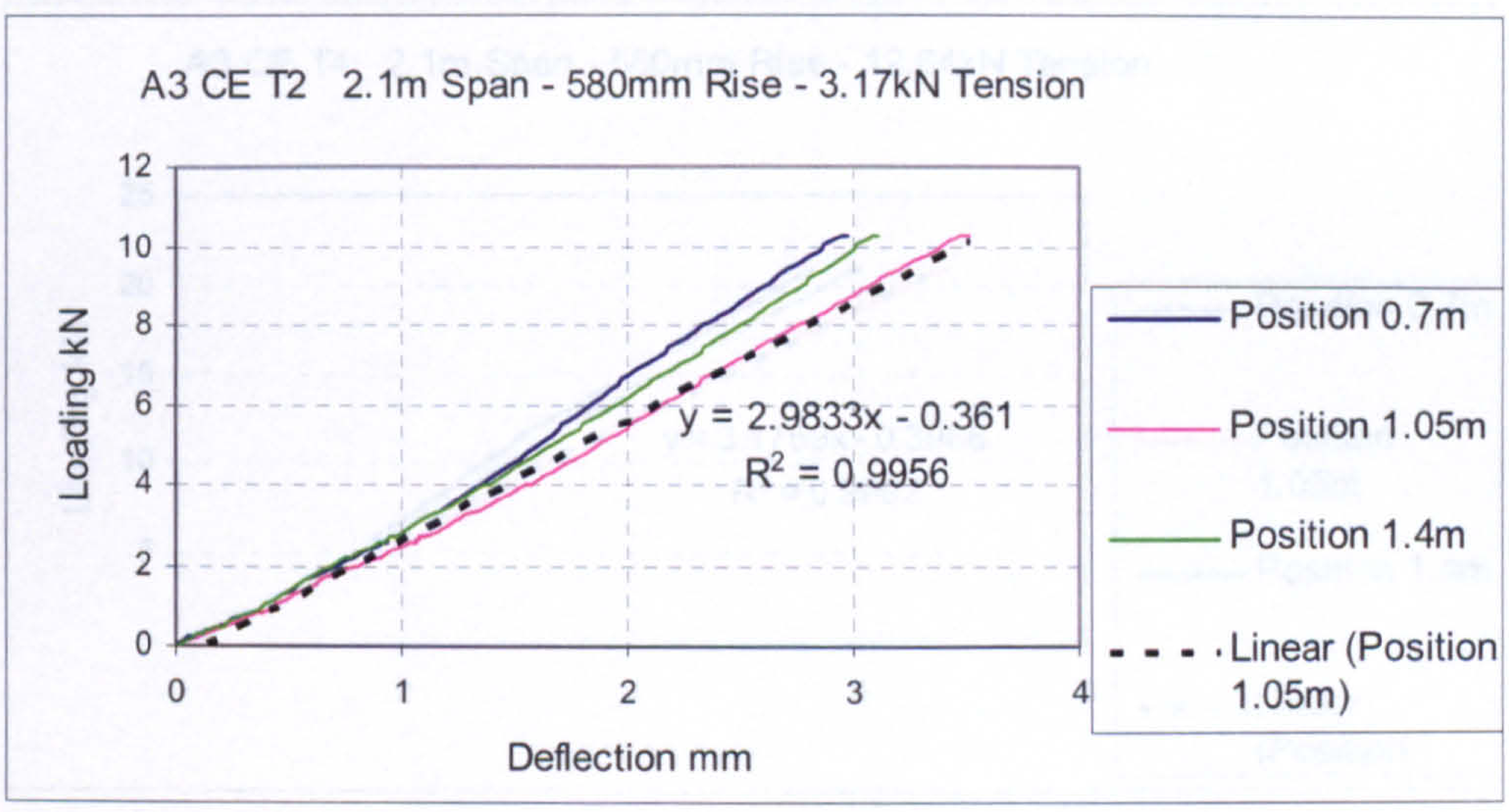


Fig 7.13b – 2.1m span – 580mm rise – 3.71kN tension

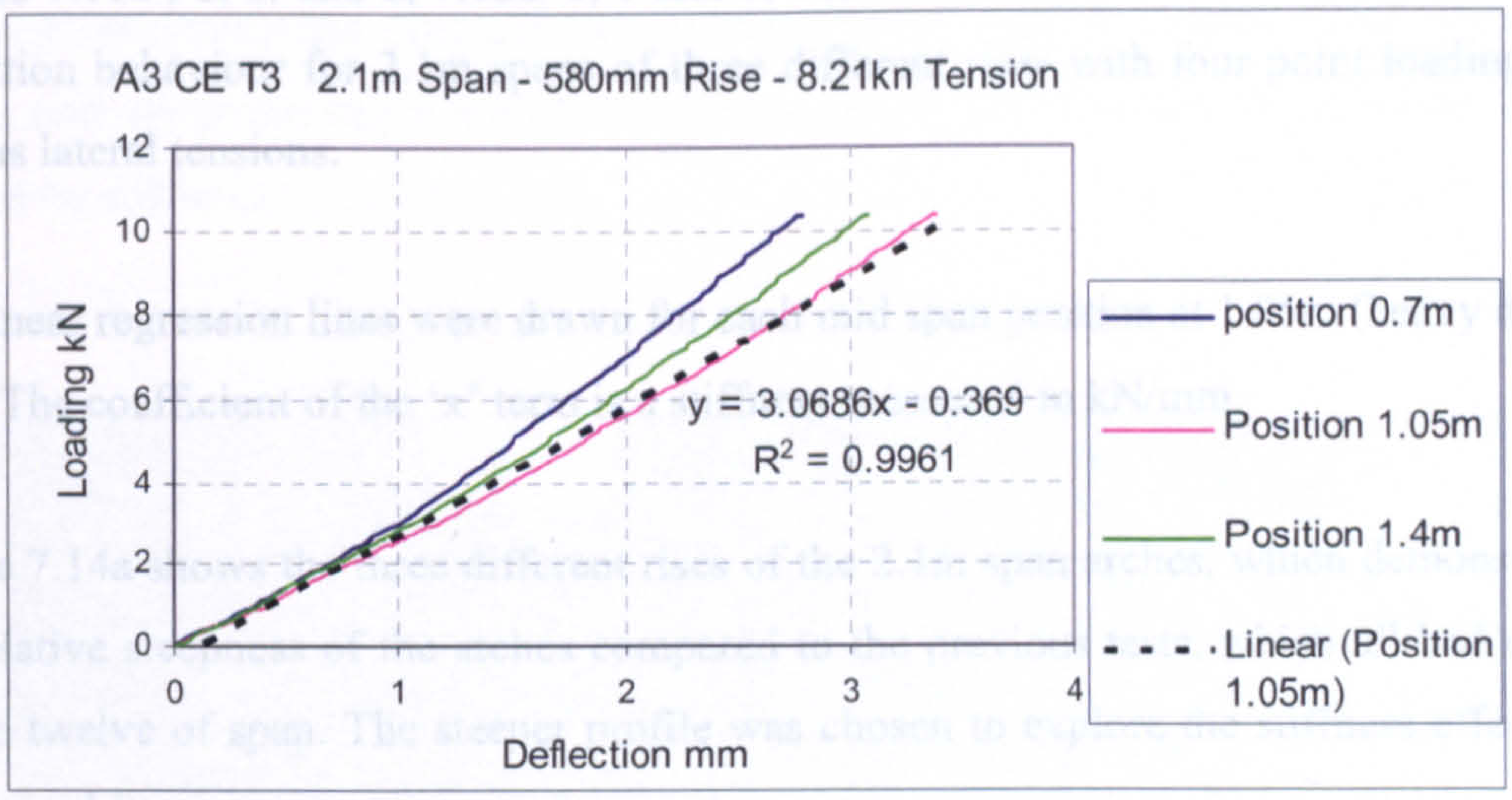


Fig 7.13c – 2.1m span – 580mm rise – 8.21kN tension

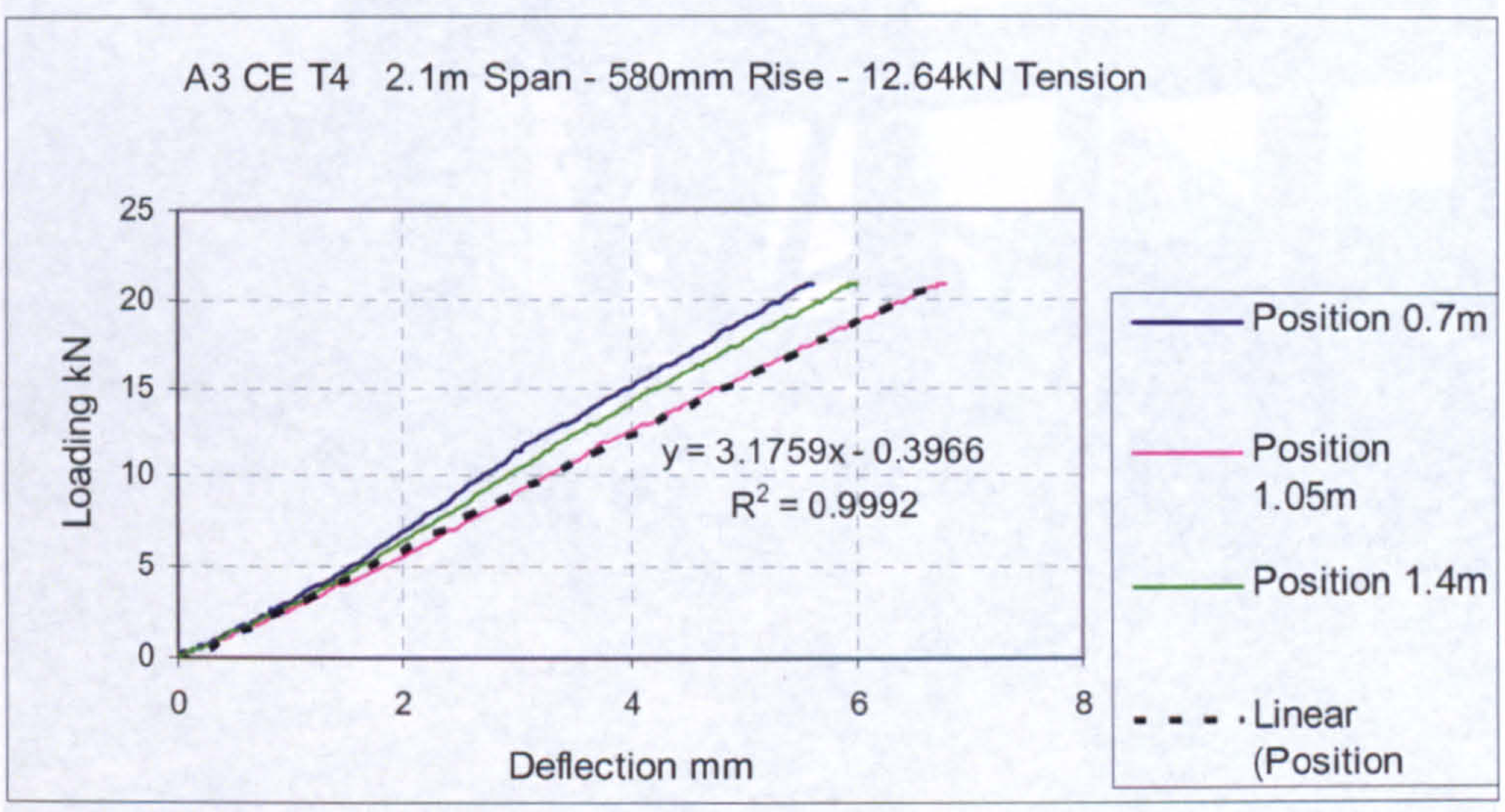
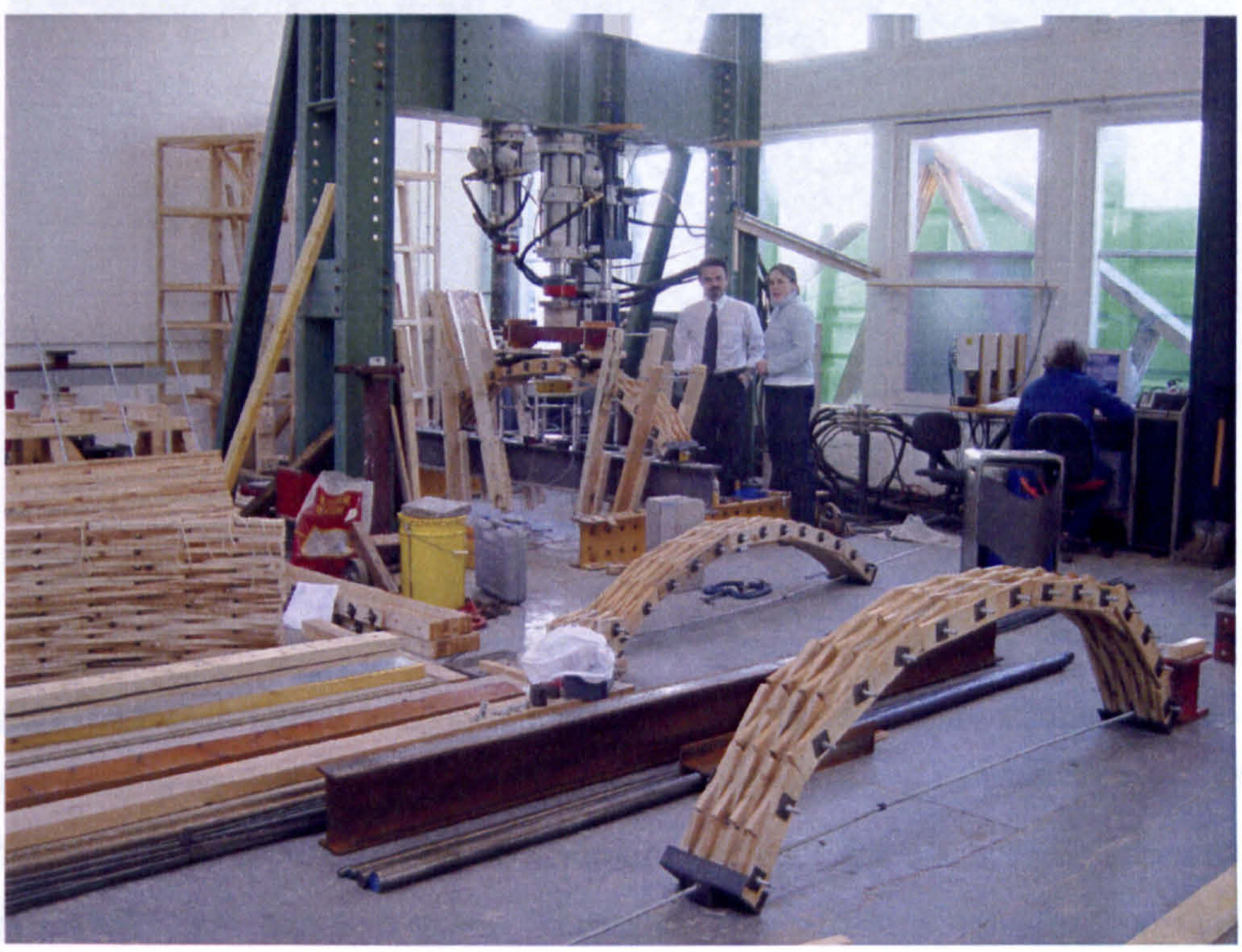


Fig 7.13d – 2.1m span – 580mm rise – 12.64kN tension

Figures 7.11a , b, c, and d, 7.12a, b, c and d, and 7.13a, b, c and d illustrate the load - deflection behaviour for 2.1m spans of three different rises with four point loading and various lateral tensions.

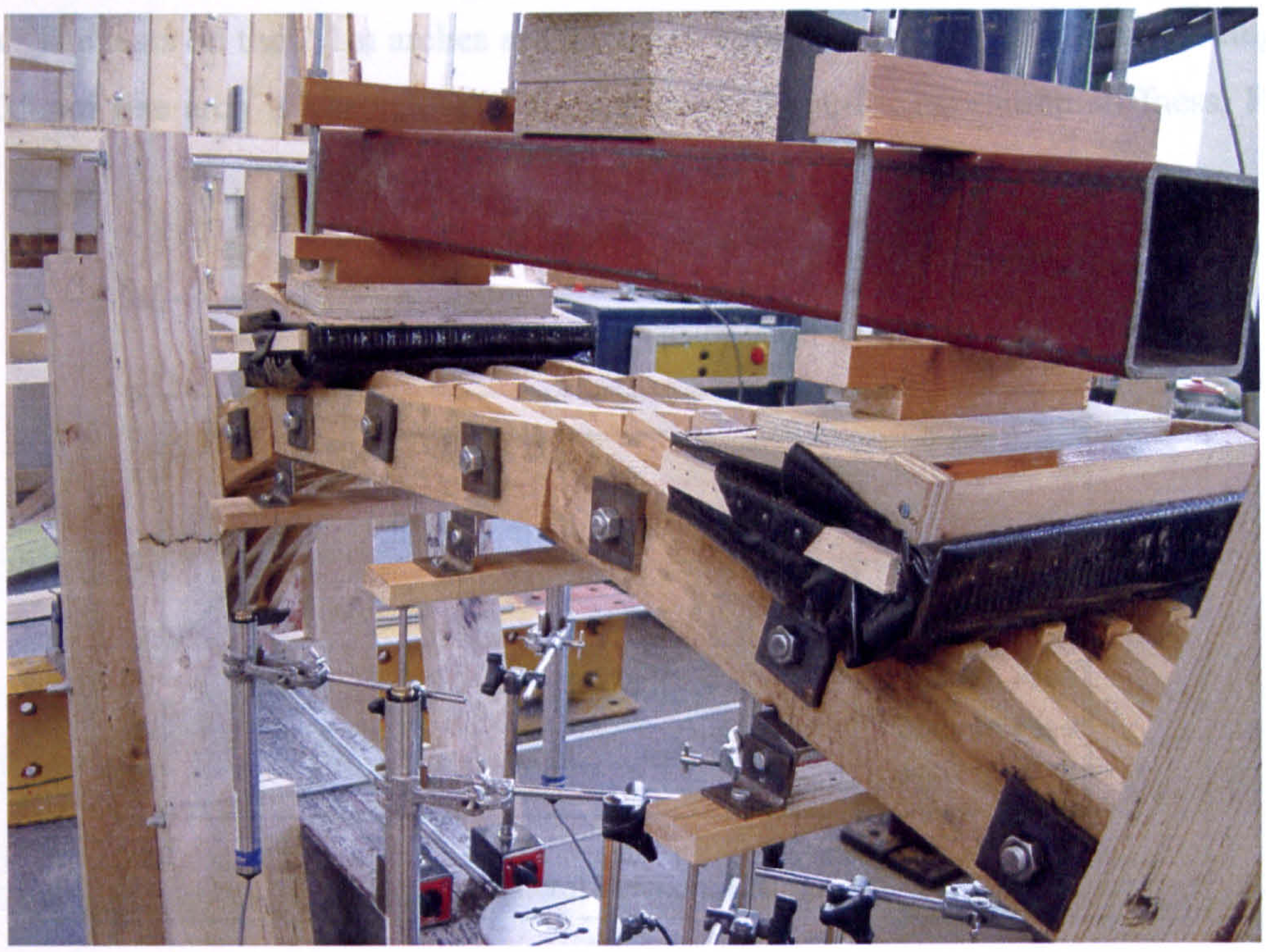
The linear regression lines were drawn for each mid span position at 1.05m (heavy dotted line). The coefficient of the 'x' term is a stiffness measured in kN/mm.

Figure 7.14a shows the three different rises of the 2.1m span arches, which demonstrates the relative steepness of the arches compared to the previous tests, which all had a unit rise to twelve of span. The steeper profile was chosen to explore the stiffness effects of different shapes.



7.14a – 2.1m span arches – three different rises

Figure 7.14b shows the threaded bar and nut fixings used to tension these laboratory arches and allow the many variations of tension necessary to test the effect of lateral tension on stiffness.



7.14b – Threaded bar and nut fixing – 2.1m span, 447mm rise

In Figure 7.15 each lateral tension is plotted against the mid span stiffness to show the effect of tension on stiffness of arches at mid span for all arches of zero, 335mm, 447mm and 580mm. It is apparent that the optimum stiffness for all rises is achieved with approximately 5kN lateral tension. Thereafter, the stiffness remains constant at a maximum value. There is also a variation in stiffness against arch rise where, again, there is an optimum which relates span, arch thickness and arch rise.

It is interesting to note from Figure 7.15 that stiffness reduces with arch rise which is the opposite of what might be expected. The span to rise ratios of the three arches are 6.2 for the 335mm rise arch, 4.7 for the 447mm rise arch and 3.6 for the 580mm rise arch. It will be shown later in Chapter 8, Figures 8.1, 8.2 and 8.3 that the optimum stiffness for timber arches occurs at an approximate span to rise ratio of 5 and reduces at ratios above and below. The tests on the 2.1m arches are therefore entirely consistent with this finding and the 335mm rise arch is very close to the optimum shape for maximum stiffness, Figure 7.17.

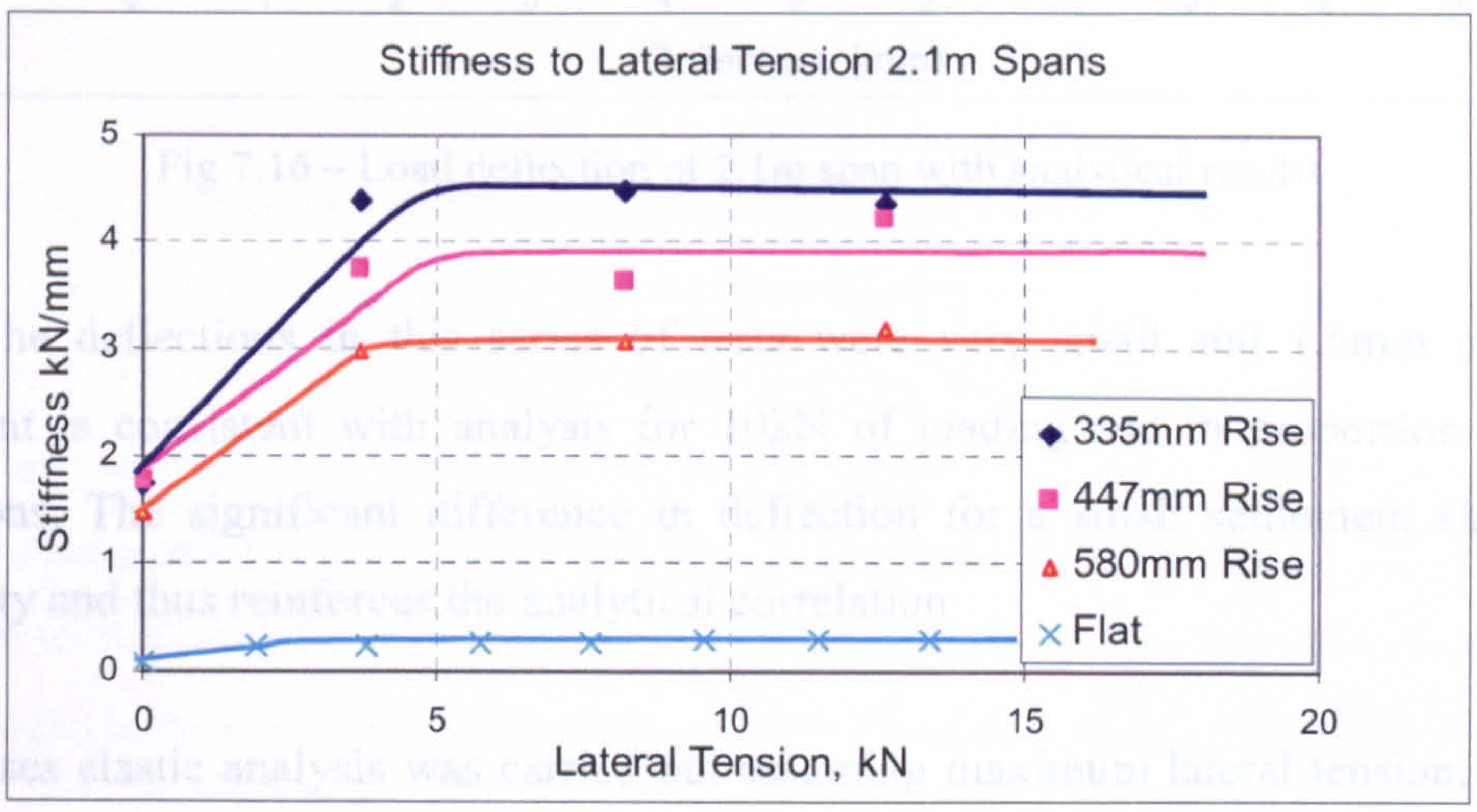


Fig. 7.15 – Arch stiffness to lateral tension for all arches

At this point it is important to show the correlation between experimental results and analytical calculations. At the end of this chapter it will be shown that the timber used for these arches had a low 'E' value of 4590N/mm^2. This was used to calculate stiffness which is plotted in Figure 7.16 as a solid heavy line. There is no settlement allowed for but some settlement will have taken place. A second stiffness is plotted which allowed for 1.5mm settlement, dashed heavy line, to show the sensitivity of the experimental data.

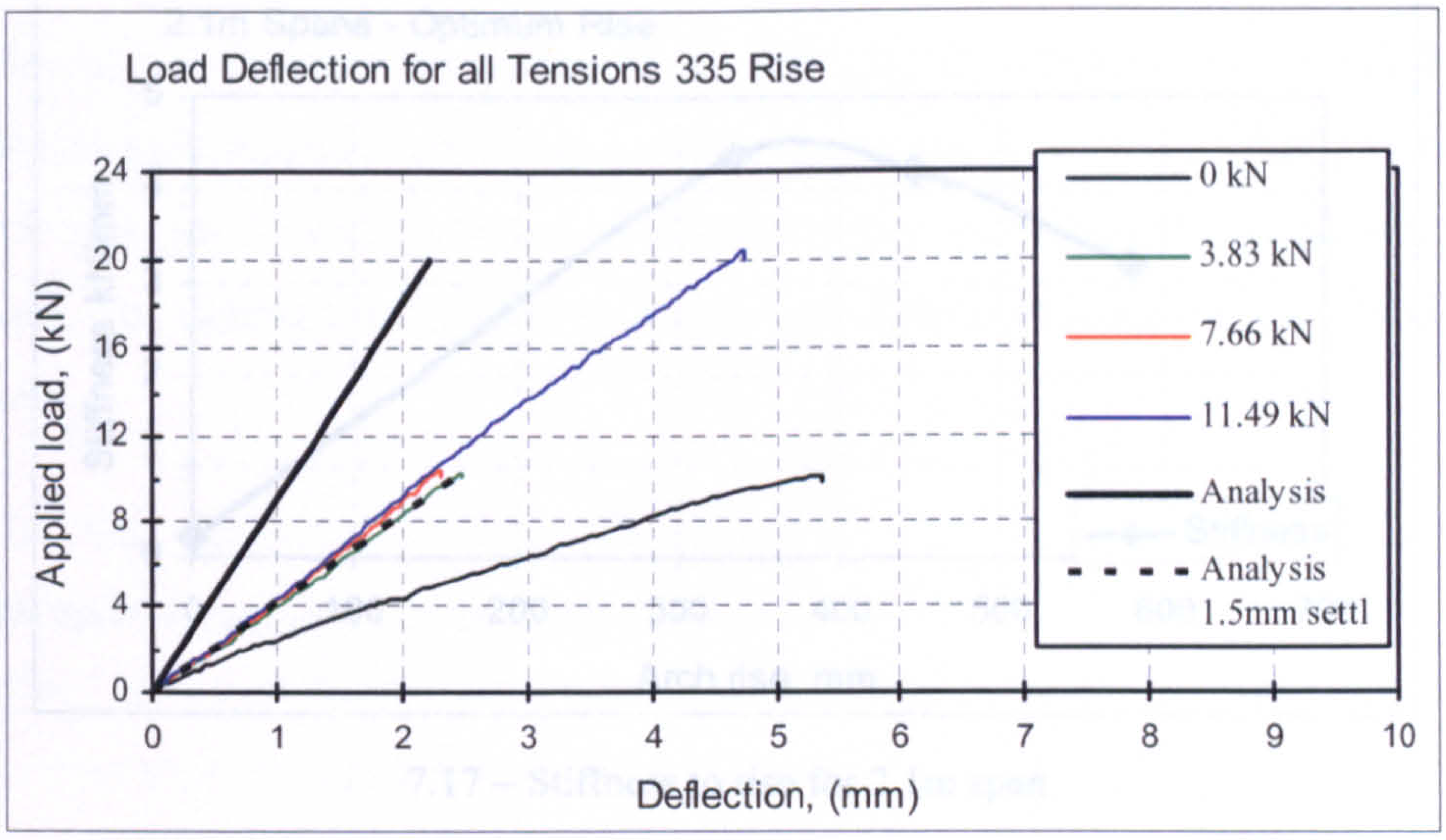
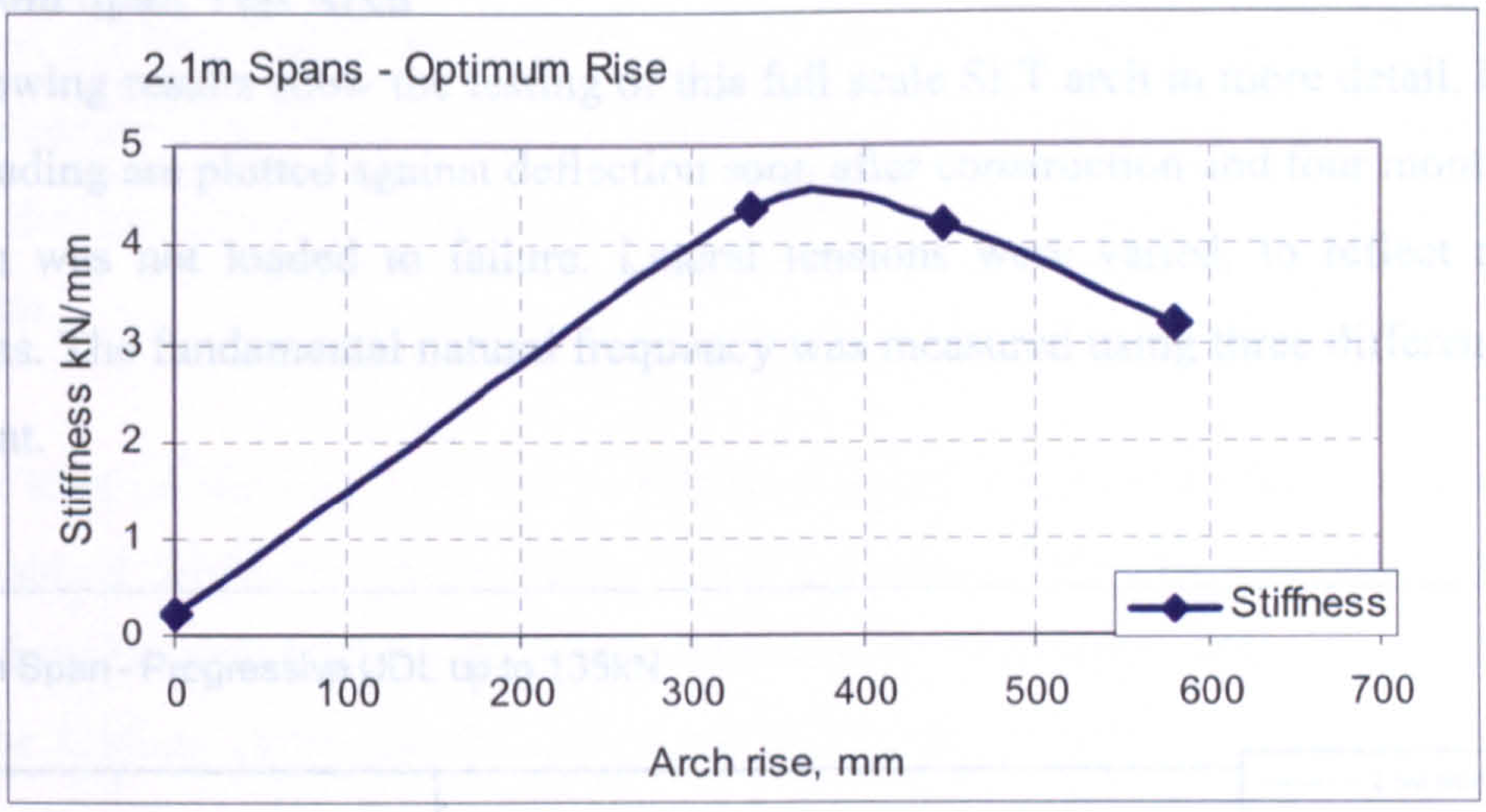


Fig 7.16 – Load deflection of 2.1m span with analytical results

All of the deflections in this series of tests were very small and 1.5mm of lateral settlement is consistent with analysis for 10kN of loading and in proportion to other deflections. The significant difference in deflection for a small settlement shows the sensitivity and thus reinforces the analytical correlation

In all cases elastic analysis was carried out assuming maximum lateral tension, and this gave good correlation for stiffness as was demonstrated in Figure 7.16. These stiffnesses were plotted against the arch rises and are shown in Figure 7.17. This shows that, from a flat profile, the span becomes stiffer but as the arch rise increases, the stiffness reduces again. The optimum rise is shown as 335mm but with more results would probably have been a little lower.

A further check on correlation between experimental load deflection results and elastic analysis was carried out by comparing stiffnesses. The values are shown in Table 8.7 and are within 20% which is considered as an acceptable experimental error.



7.17 – Stiffness to rise for 2.1m span

The final tests carried out on the 2.1m span bridges were under quarter point loadings, to failure. As with all other quarter point loadings, the opposite side of the arch rose and the maximum deflection and bending moment occurred under the applied load. All 2.1m span arches were deemed to have failed at 25kN when deflections under the load were between 16mm and 18mm. This give stiffnesses between 1.4kN/mm and 1.5kN/mm. This compares with stiffnesses of greater than 3kN/mm for 4 point loading thus showing that arches do not have as much load bearing capacity if loaded at the quarter point.

Failure was determined when a laminate split. In practice, a full scale arch would continue past this point and still take load until a number of other laminates failed. This is a very useful feature of a timber arch.

7.4.1 Conclusions from 2.1m Span Tests

The experimental behaviour of the 2.1m span arches has shown good correlation with elastic analysis. The stiffness at the quarter point, with load at that position, is smaller and may be a governing design criterion. The effect of lateral tension is clear and shows that sufficient friction is provided at low tensions for transfer of load. This shows the built-in safety of stress laminated arch structures. Finally the tests clearly indicate that there is an optimum rise for a particular span which could be a useful design tool therefore this should be evaluated further.

7.5 20m Span Test Arch

The following results show the testing of this full scale SLT arch in more detail. Loading and unloading are plotted against deflection soon after construction and four months later. The arch was not loaded to failure. Lateral tensions were varied, to reflect real site conditions. The fundamental natural frequency was measured using three different sets of equipment.

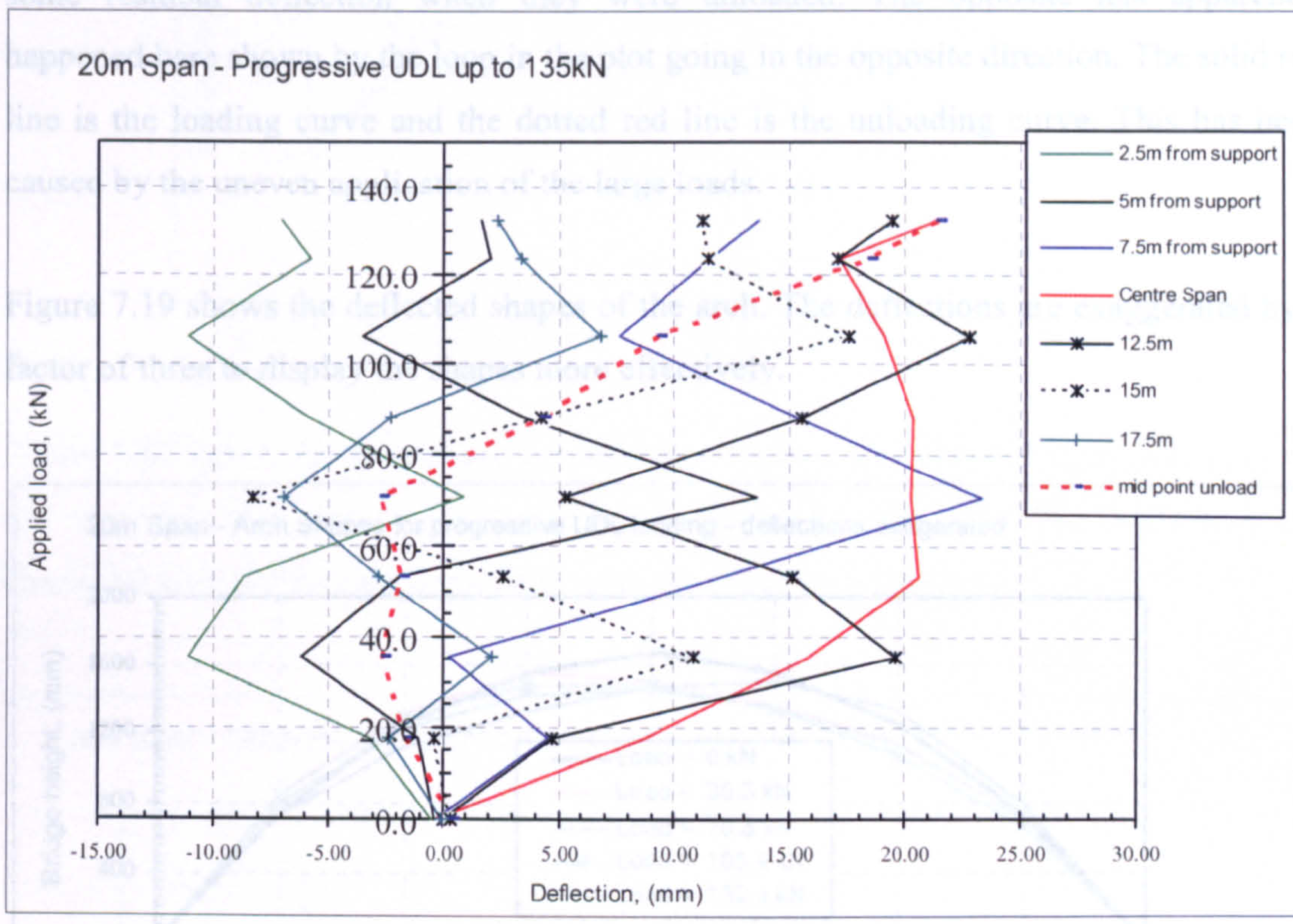


Fig 7.18 – Load deflection for 20m span and UDL

Figure 7.18 shows erratic deflections at all points along the arch for a UDL, up to the maximum load. This was expected for a large structure where some relaxation in lateral tension had taken place since the time it was originally built. These results are a realistic reflection of a real structure, especially given that the UDL consisted of very large individual loads applied at various positions. The central deflection increased rapidly at the beginning because that is where the loads were applied first and this caused negative deflections at points nearer the supports. The average stiffness at the centre, for this initial loading situation, can be given by the trendline as 4.9kN/mm, but a more realistic value may be the maximum load divided by the maximum deflection 135/21.41, which is 6.3kN/mm.

Previous tests on the 6m, 15m spans, Figures 7.1 and 7.5, showed the arches retaining some residual deflection when they were unloaded. The opposite has apparently happened here shown by the loop in the plot going in the opposite direction. The solid red line is the loading curve and the dotted red line is the unloading curve. This has been caused by the uneven application of the large loads.

Figure 7.19 shows the deflected shapes of the arch. The deflections are exaggerated by a factor of three to display the shapes more effectively.

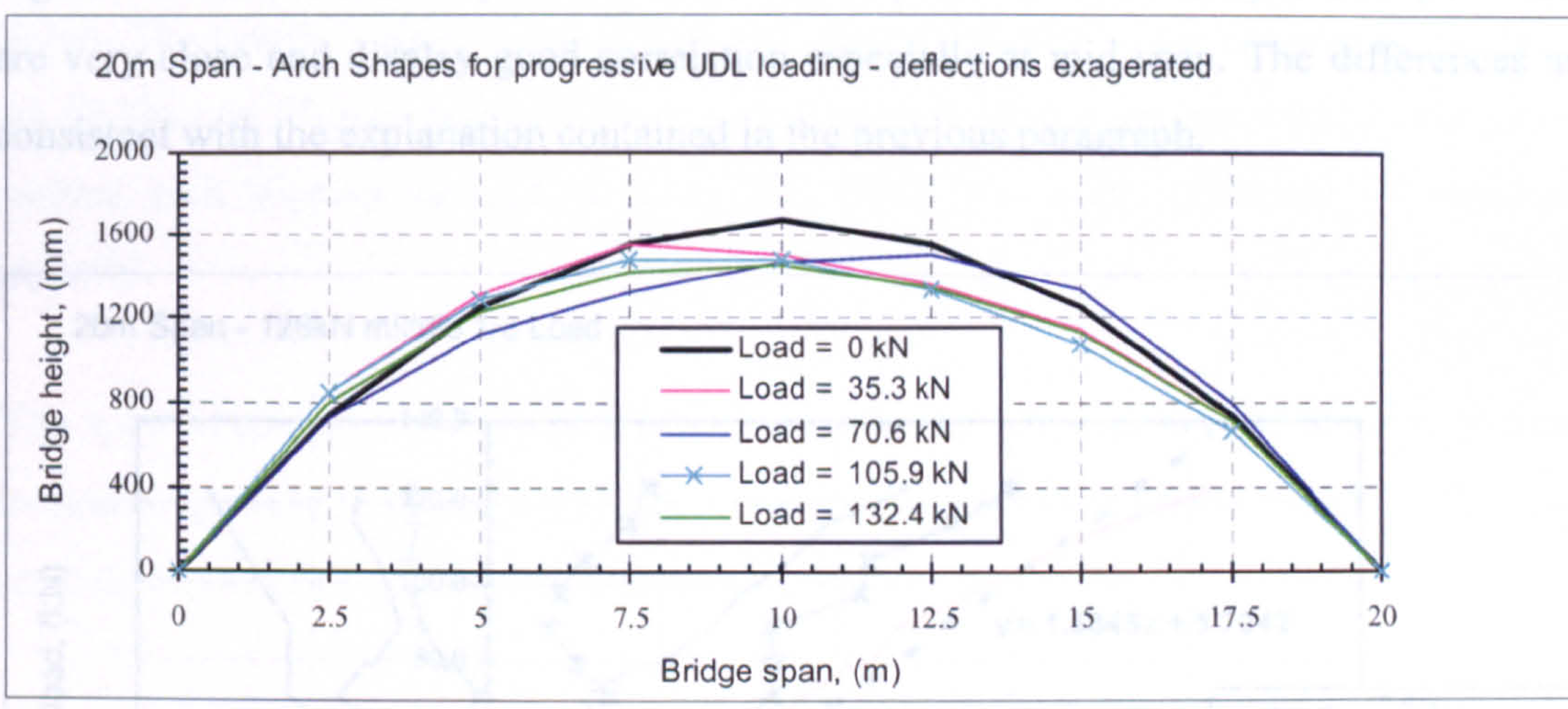


Fig 7.19 – 20m span arch shape for UDL

It can be seen that the deflected shape is not uniform under uniform load, well illustrated by the crossing of the 70.6kN and 132.4kN load shapes. This shows that some of the asymmetric deformation from the initial loading remained even at full design load. This occurred because large loads were placed at single locations on the deck all at one time producing high local deflections. Because of the flexibility of the stress lamination it was considered that the structure would recover when unloaded and left to settle and subsequent loadings would give a more consistent result. The effect was interesting to note and would not be expected from a steel or concrete structure.

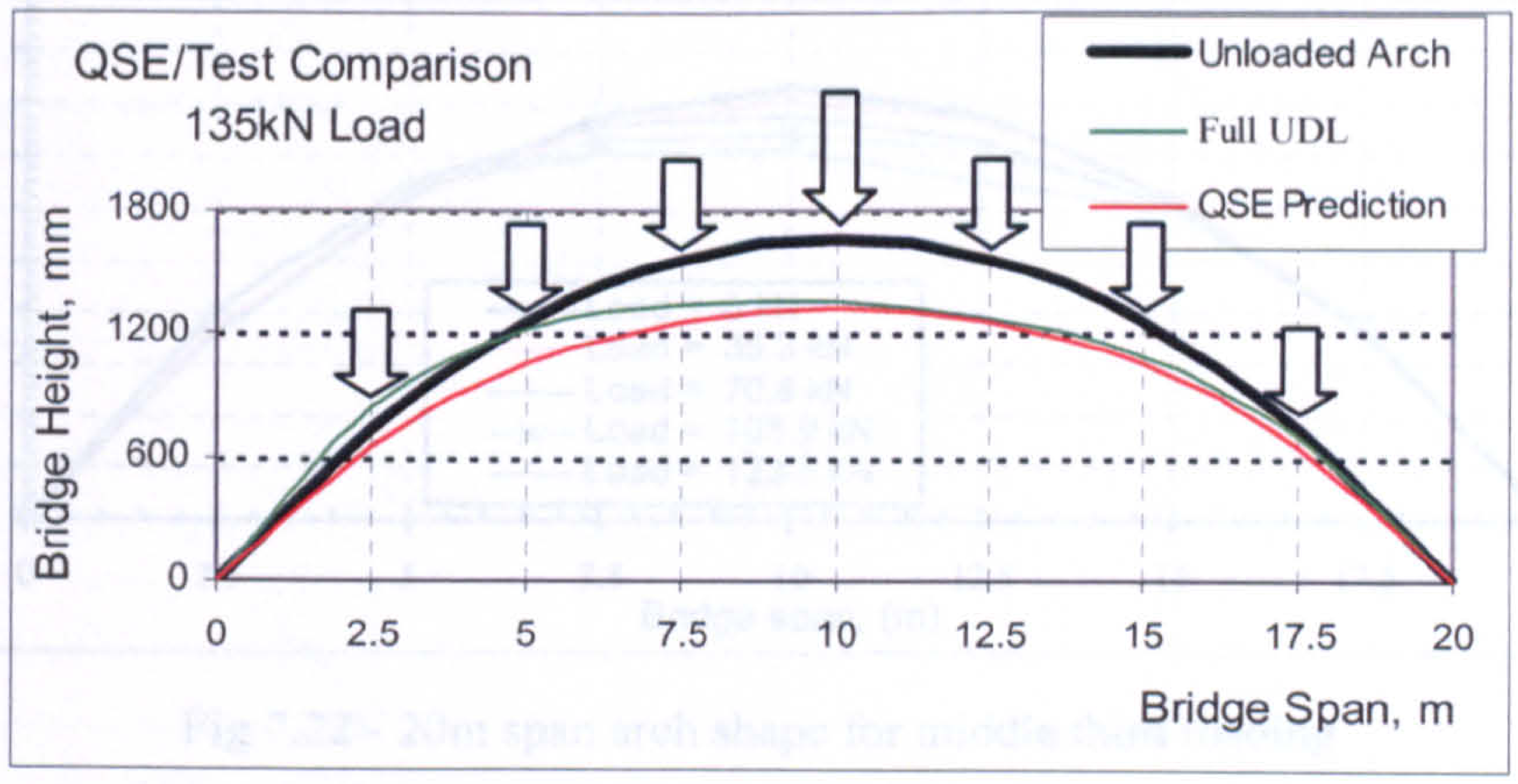


Fig 7.20 – 20m span arch shape for UDL showing analytical results

Figure 7.20 shows the comparison between the experimental and analytical shapes which are very close and display good correlation especially at mid span. The differences are consistent with the explanation contained in the previous paragraph.

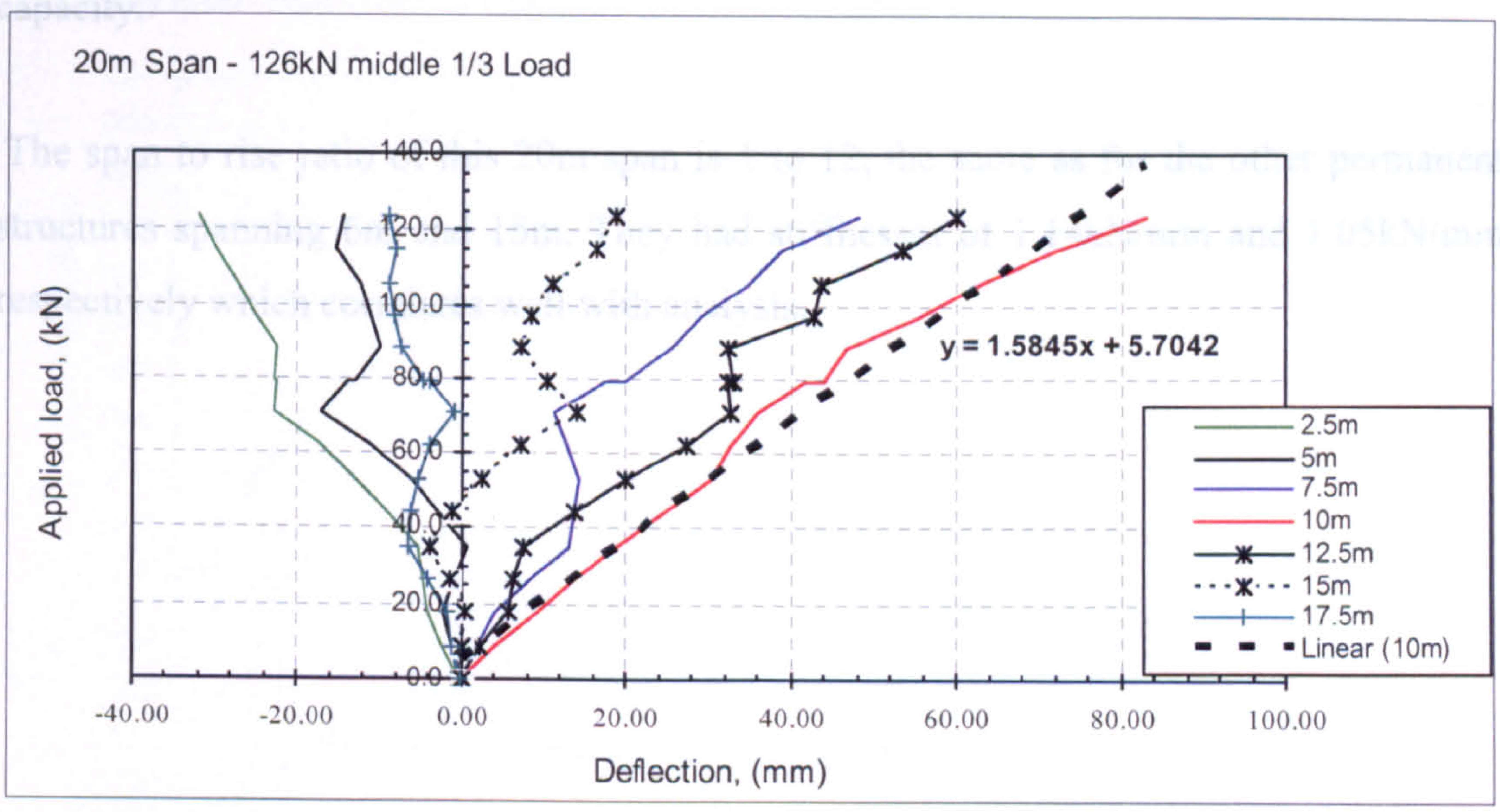


Fig 7.21 – Load deflection for 20m span under middle third loading

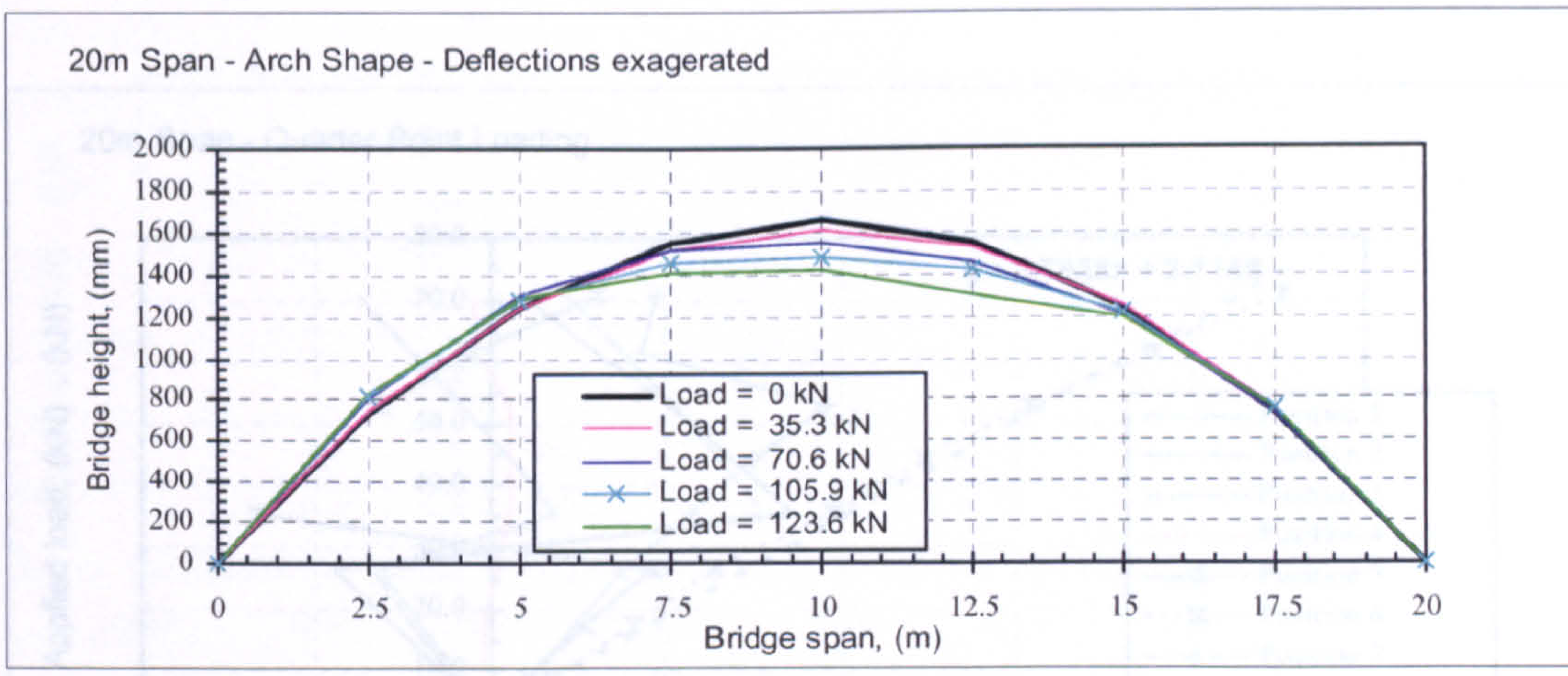


Fig 7.22 - 20m span arch shape for middle third loading

Figures 7.21 and 7.22 show the load deflection behaviour for a middle third loading case. This is comparable to the four point loading used in the laboratory for the small scale arches. The trendline shows a much more consistent measure of the stiffness at the centre as1.58 kN/mm. This greater consistency was due to it being a second loading after initial internal arch slippage had taken place. The result was a true reflection of structural capacity.

The span to rise ratio of this 20m span is 1 to 12, the same as for the other permanent structures spanning 6m and 15m. They had stiffnesses of 1.14kN/mm and 1.05kN/mm respectively which correlates well with analysis.

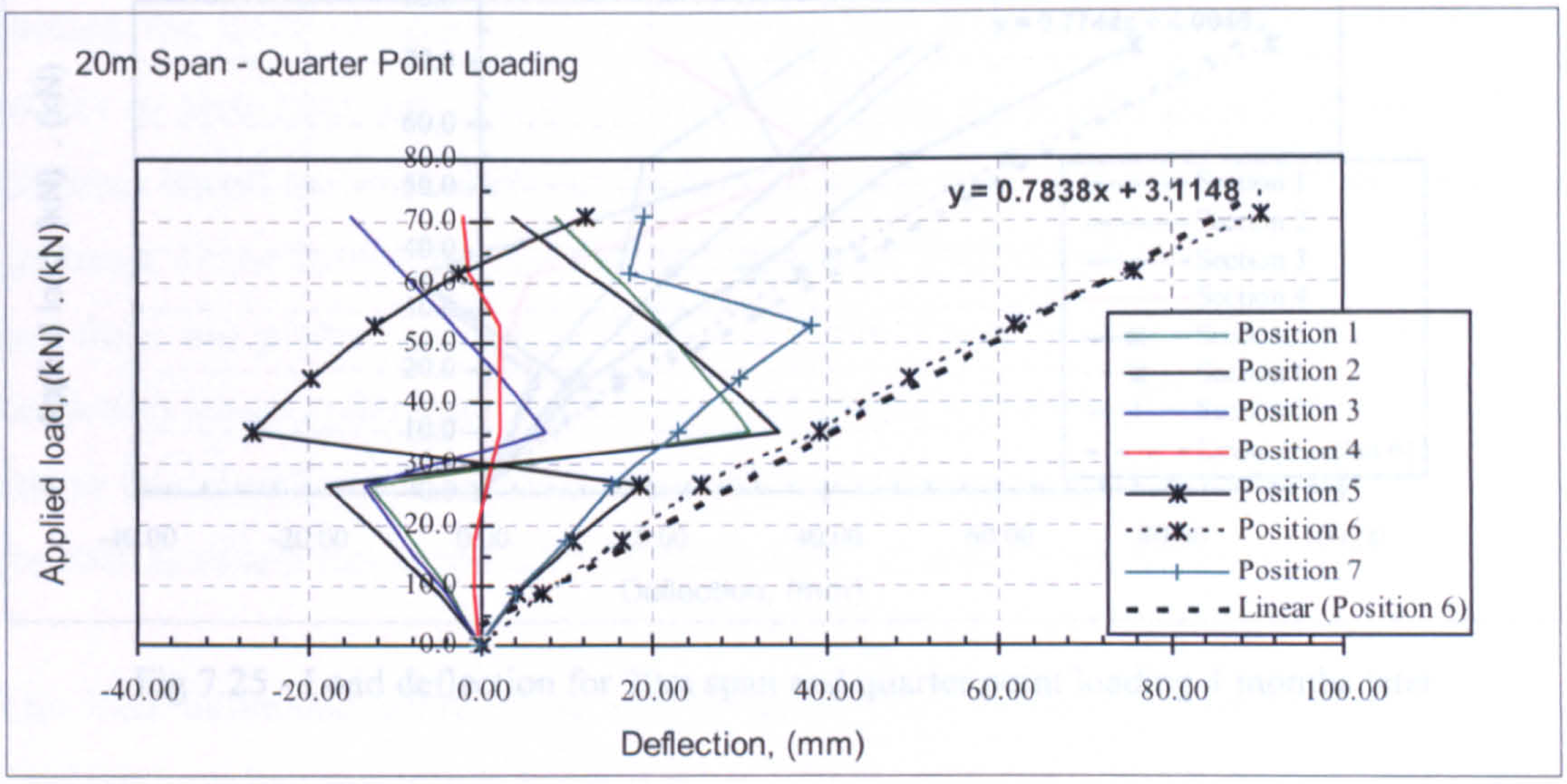


Fig 7.23 - Load deflection for 20m span and quarter point loading

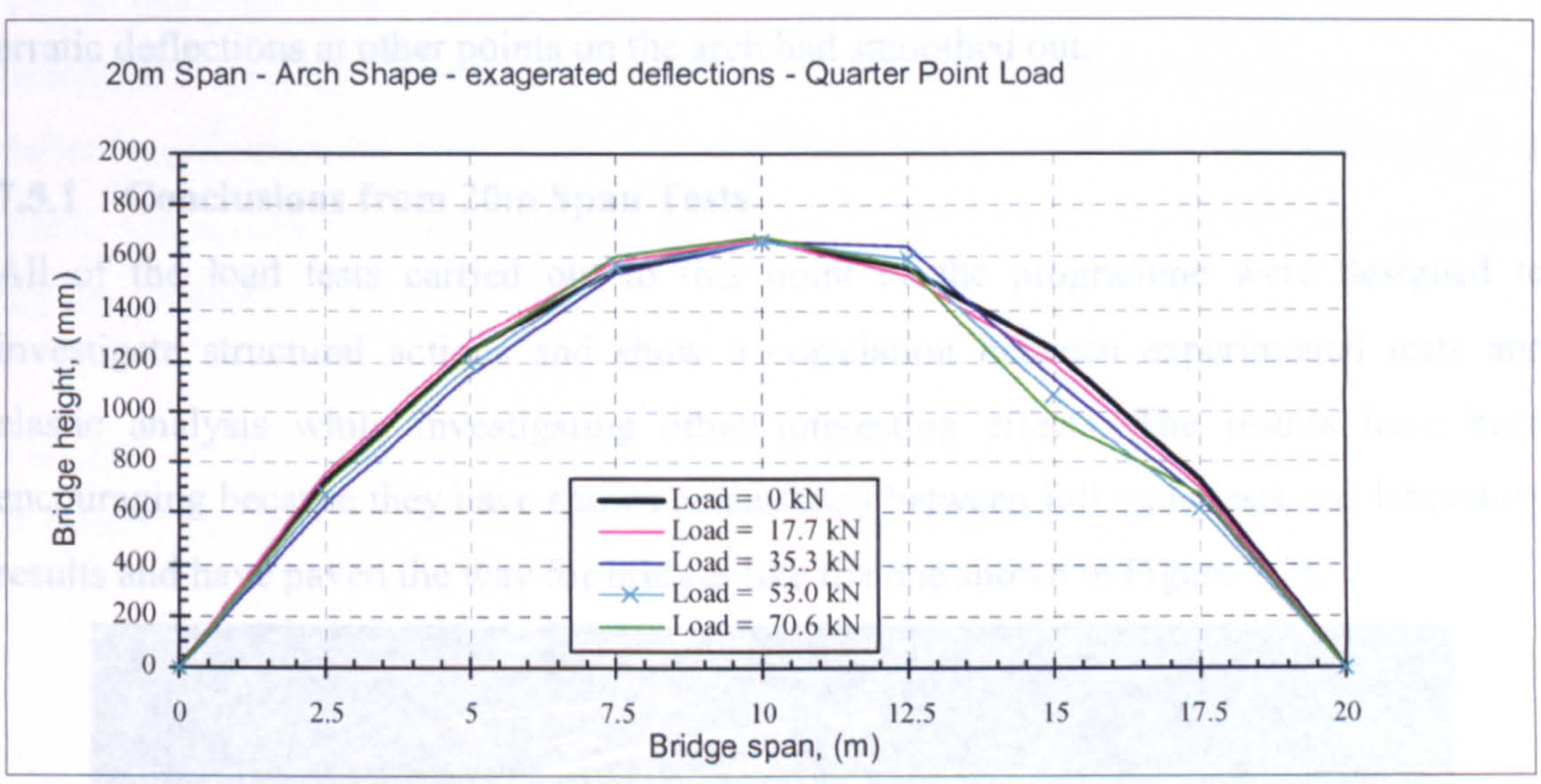


Fig 7.24 - 20m span arch shape for quarter point loading

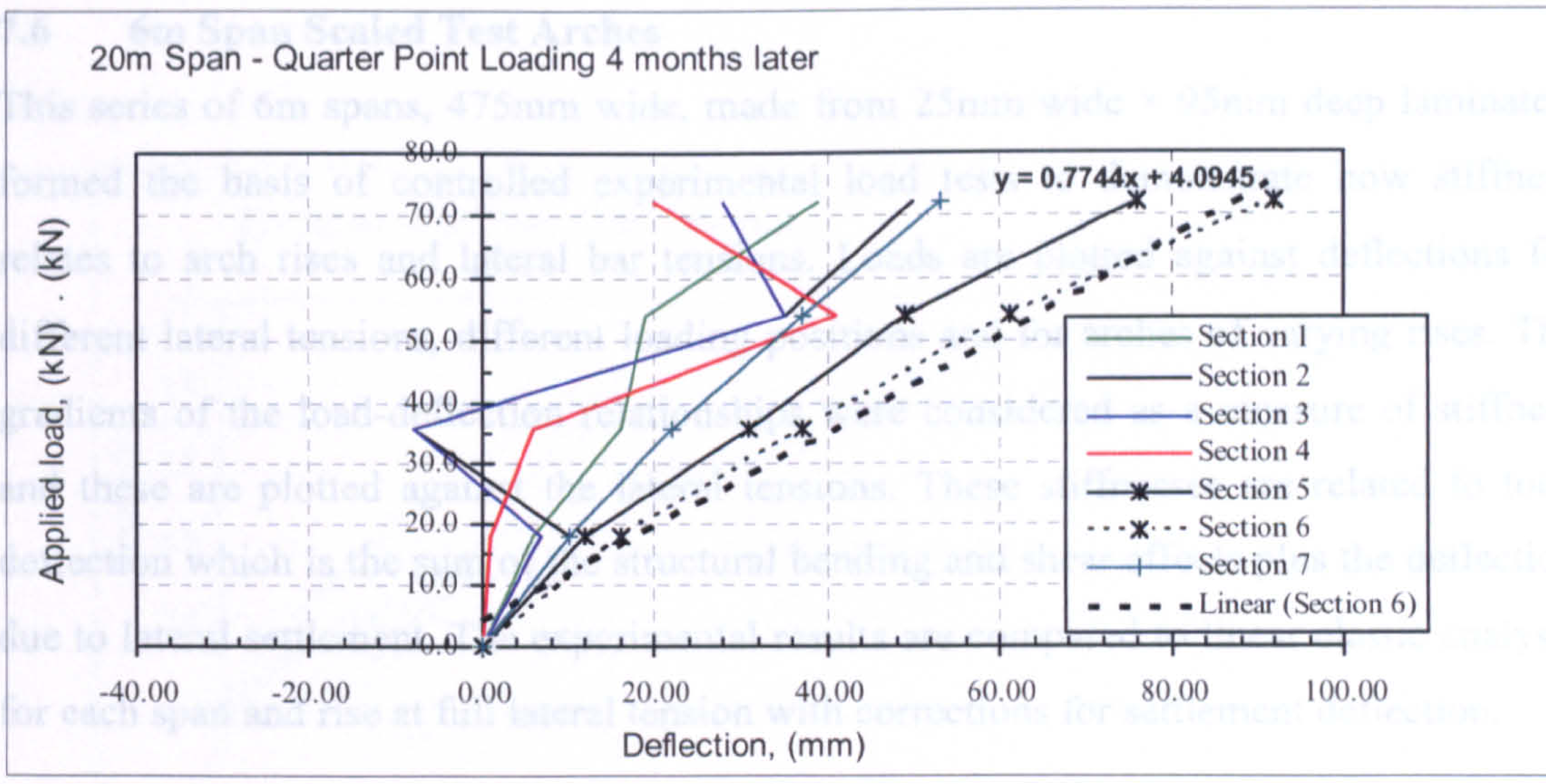


Fig 7.25 - Load deflection for 20m span and quarter point loading 4 months later

Figures 7.23, 7.24 and 7.25 show the same load deflection plots as previous 20m span charts only, this time, with loading at the quarter point. In this situation the stiffness is considerably reduced, to 0.78 kN/mm. As mentioned before it was interesting that after four months and with some re-tensioning, the stiffness remained the same but the more erratic deflections at other points on the arch had smoothed out.

7.5.1 Conclusions from 20m Span Tests

All of the load tests carried out to this point of the programme were designed to investigate structural actions and show a correlation between experimental tests and elastic analysis while investigating other interesting effects. The results have been encouraging because they have shown consistency between full scale tests and laboratory results and have paved the way for bridges like the one shown in Figure 7.26.

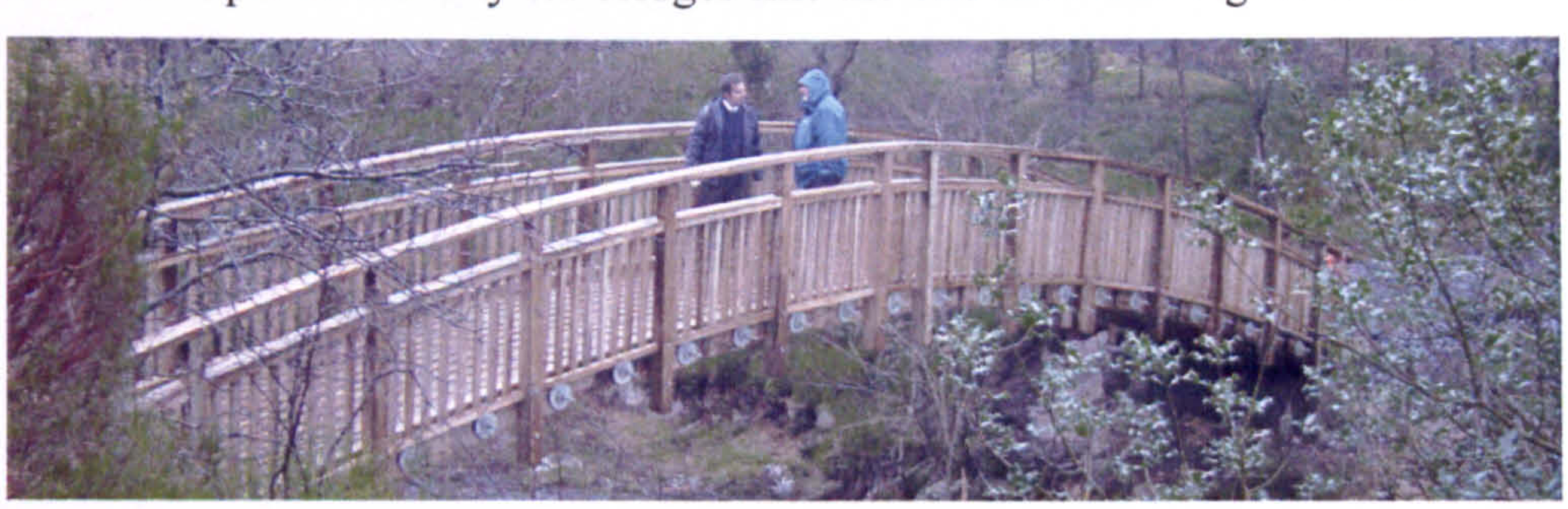


Fig 7.26 – 20m span SLT arch Queen Elizabeth Forest Park

7.6 6m Span Scaled Test Arches

This series of 6m spans, 475mm wide, made from 25mm wide × 95mm deep laminates, formed the basis of controlled experimental load tests to demonstrate how stiffness relates to arch rises and lateral bar tensions. Loads are plotted against deflections for different lateral tensions, different loading positions and for arches of varying rises. The gradients of the load-deflection relationships were considered as a measure of stiffness and these are plotted against the lateral tensions. These stiffnesses are related to total deflection which is the sum of the structural bending and shear effects plus the deflection due to lateral settlement. The experimental results are compared to linear elastic analysis for each span and rise at full lateral tension with corrections for settlement deflection.

The load deflection plots all recorded loading and unloading. The unloading curves displayed temporary residual deflections which dissipate with time. At zero lateral tension the arches display very low stiffness and non linear structural action which cannot be used to predict load capacities.

The arches were loaded according to their capacity which depends on span rise ratio and lateral tensions. Fore example the flat slab could only take just over 5kN with a very large deflection whereas the strongest arch with the maximum lateral tension was loaded up to 20kN. The arches were not loaded to failure but were loaded to approximately four times the design load.

The plots are shown in Figures. 7.27a, b c & d, 7.28a, b, c & d, 7.29a, b, c & d, 7.29a, b, c & d and 7.30a, b, c & d for flat, 250mm rise, 500mm rise and 1000mm rise respectively.

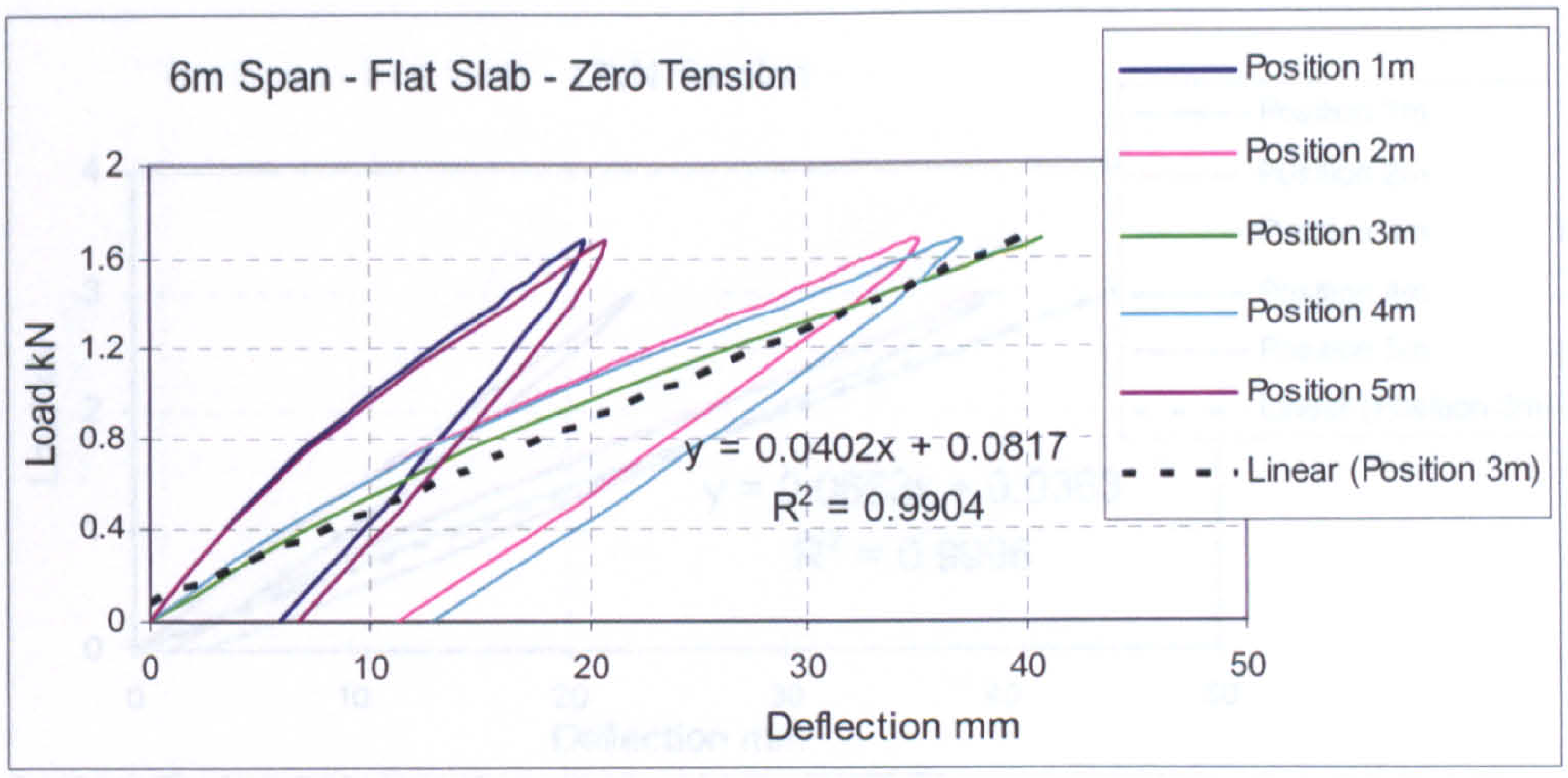


Fig 7.27a – 6m span – flat – zero tension

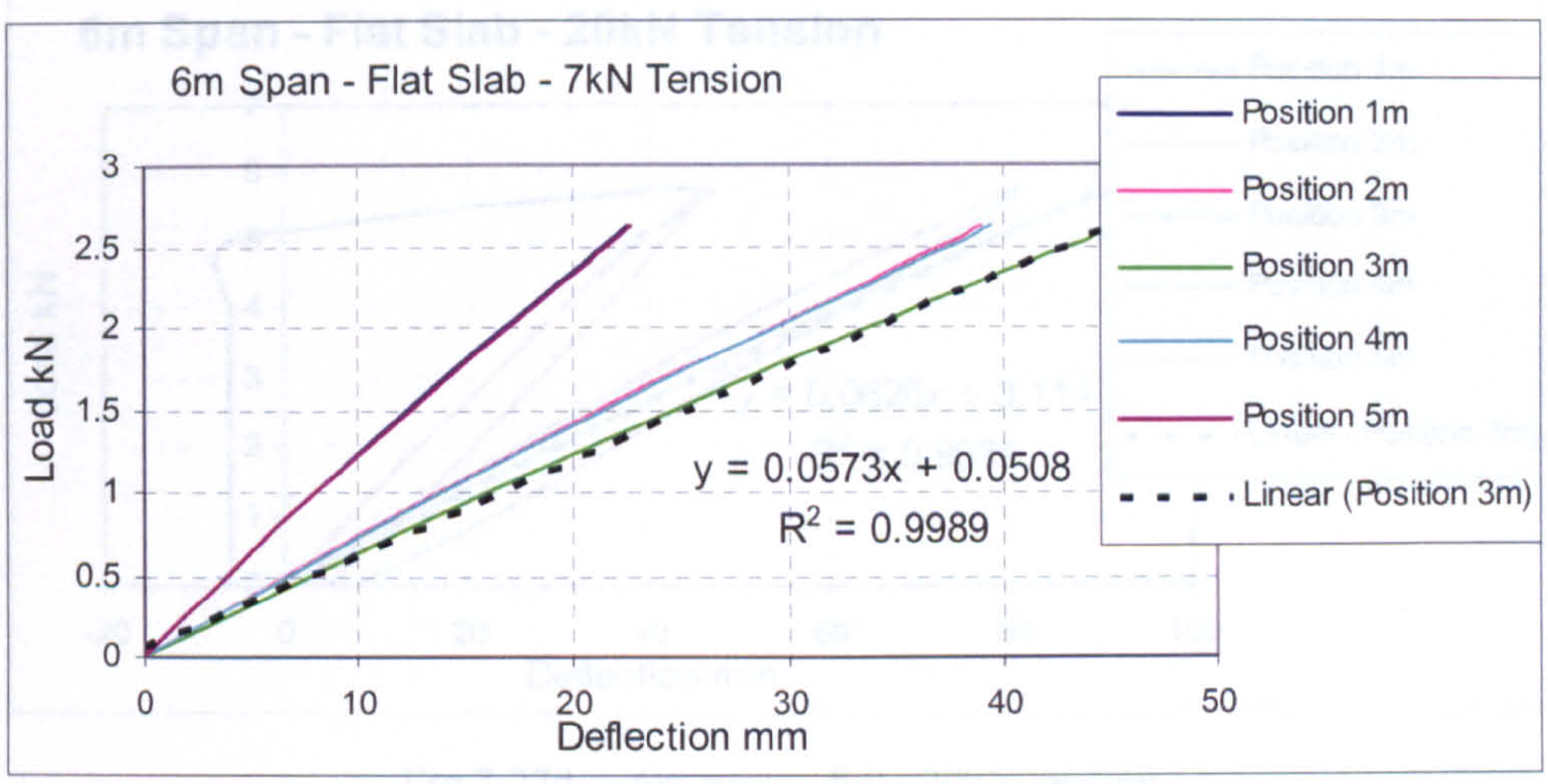


Fig 7.27b – 6m span – flat – 7kN tension

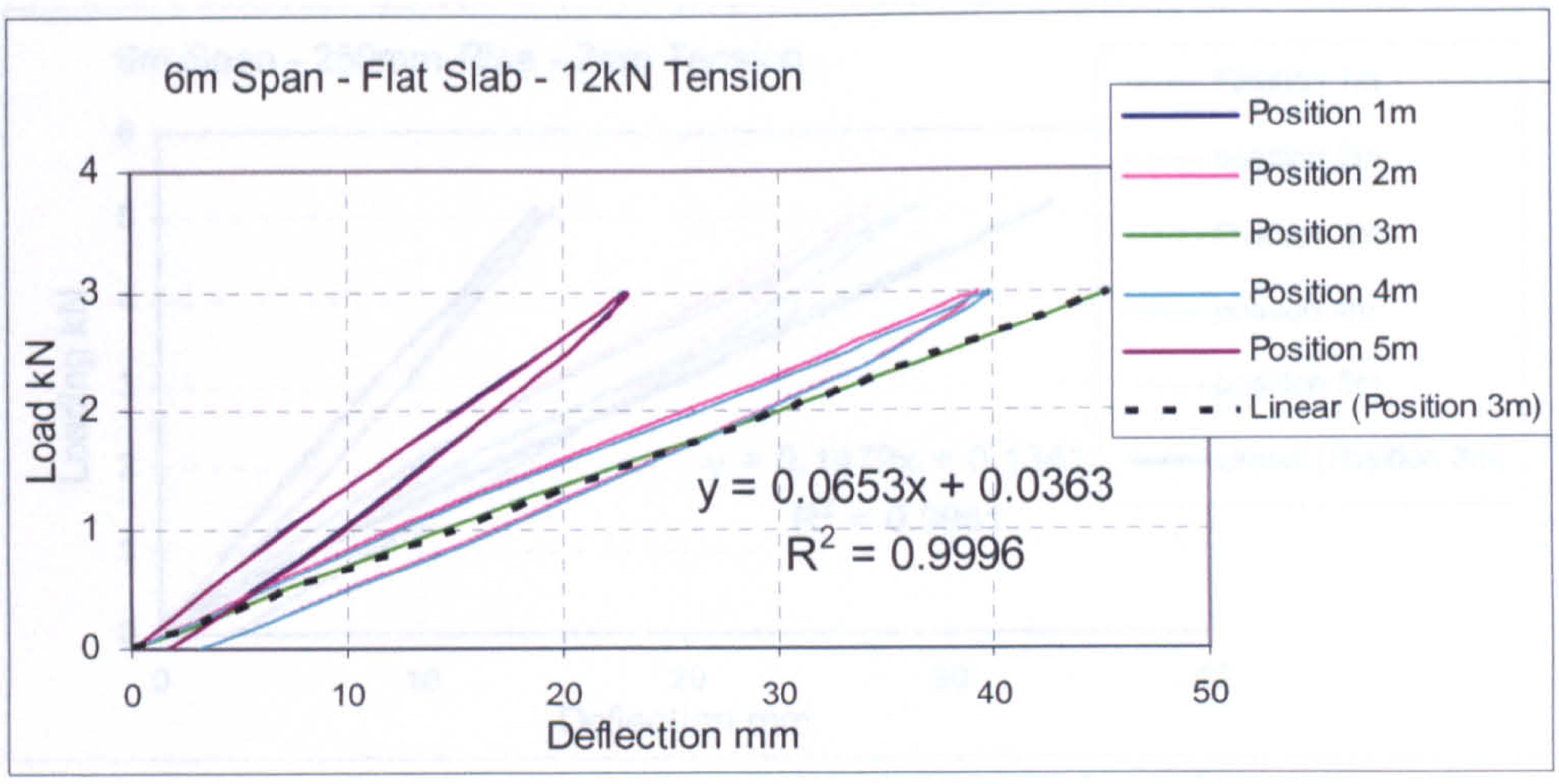


Fig 7.27c – 6m span – flat –12kN tension

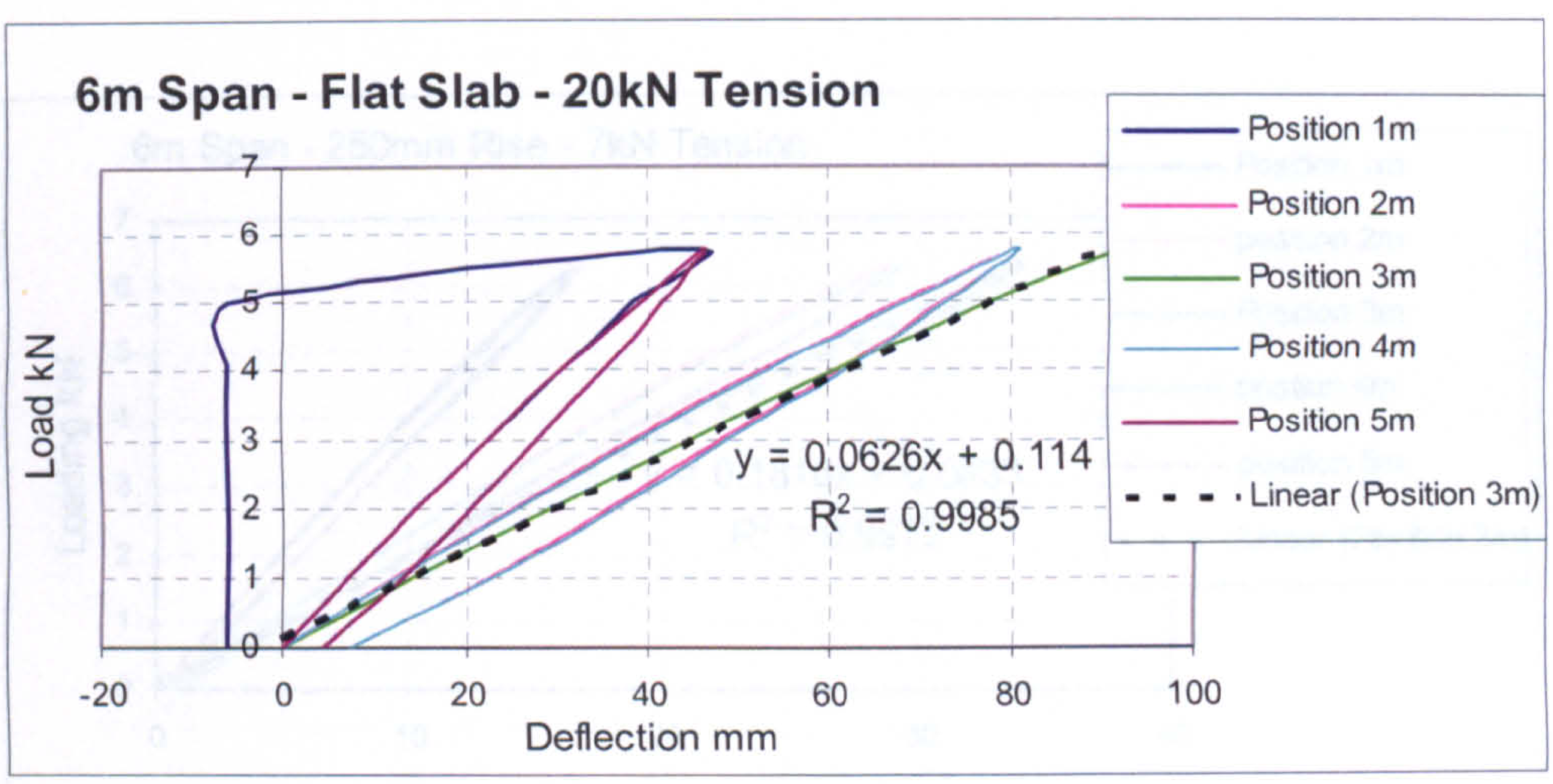


Fig 7.27d – 6m span – flat –20kN tension

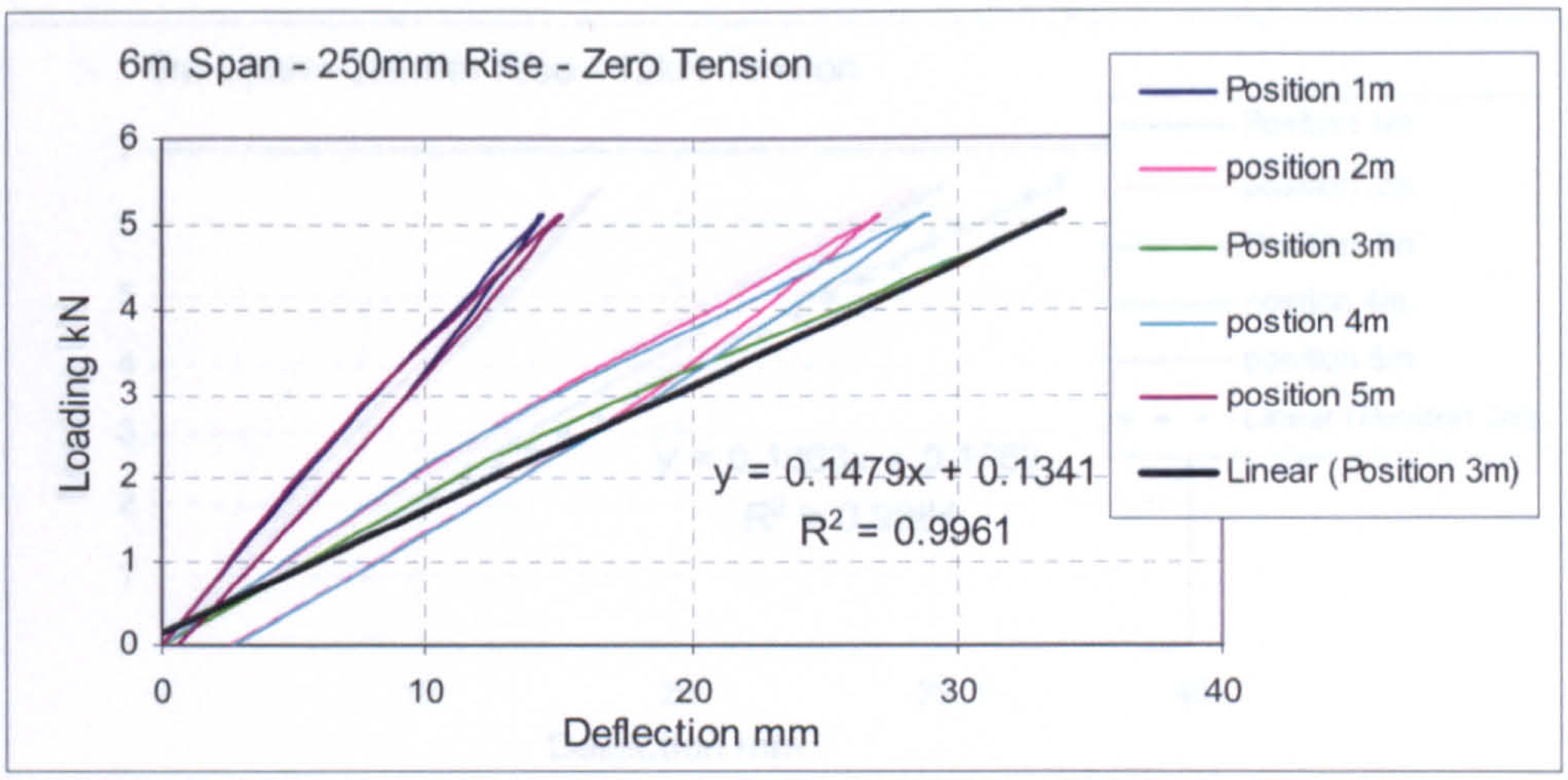


Fig 7.28a – 6m span – 250mm rise – zero tension

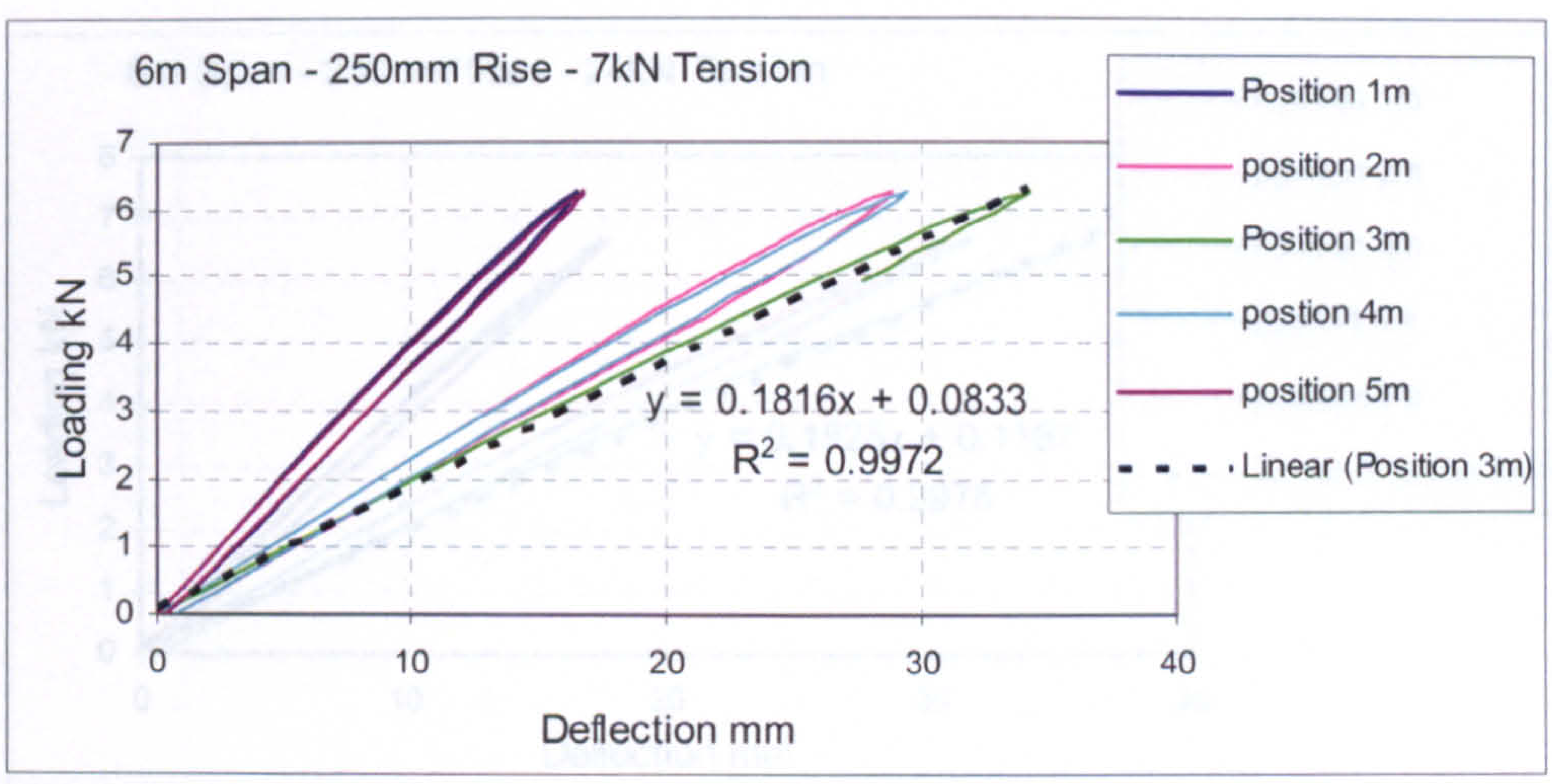


Fig 7.28b – 6m span – 250mm rise – 7kN tension

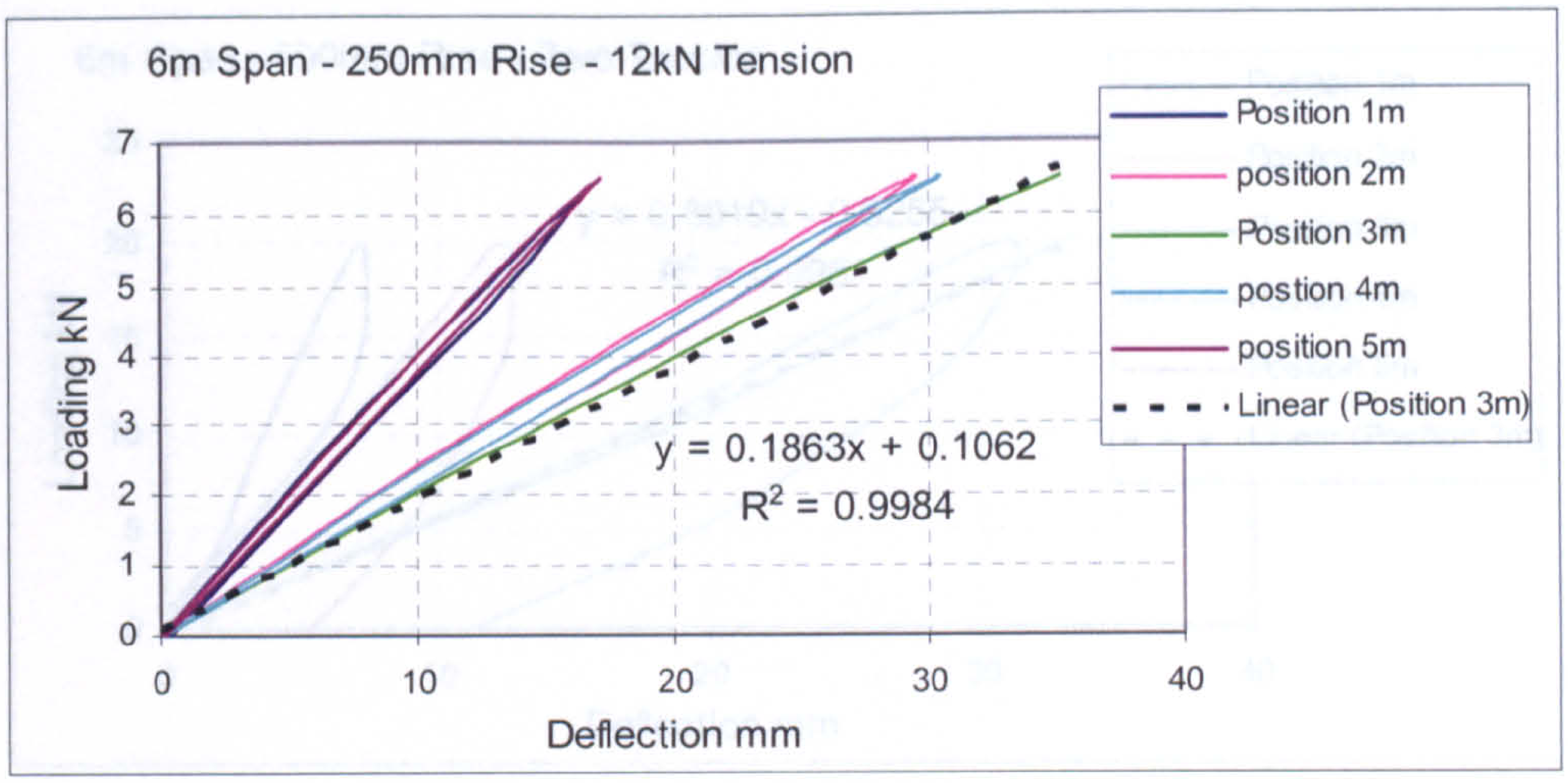


Fig 7.28c – 6m span – 250mm rise – 12kN tension

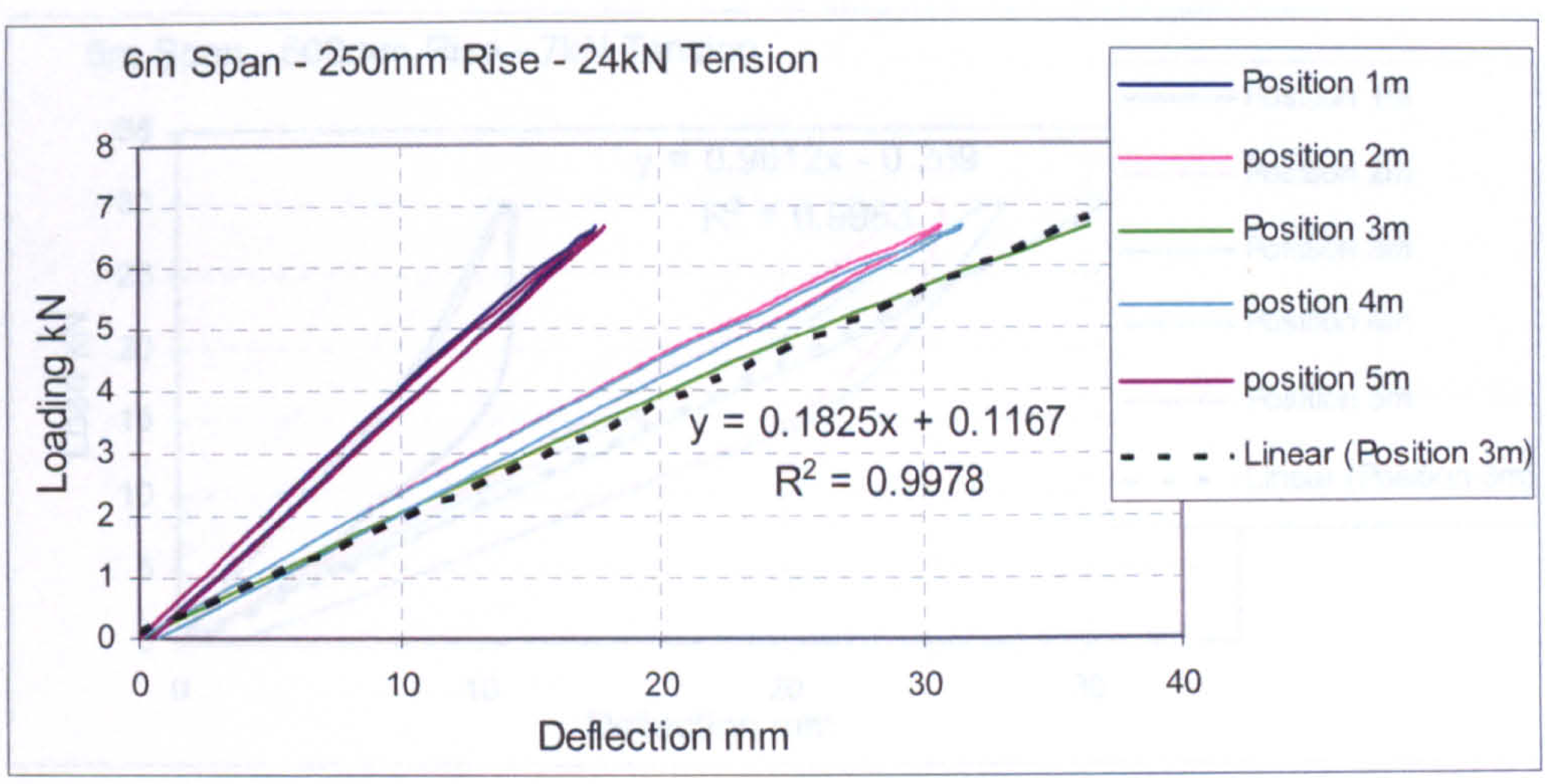


Fig 7.28d – 6m span – 250mm rise – 24kN tension

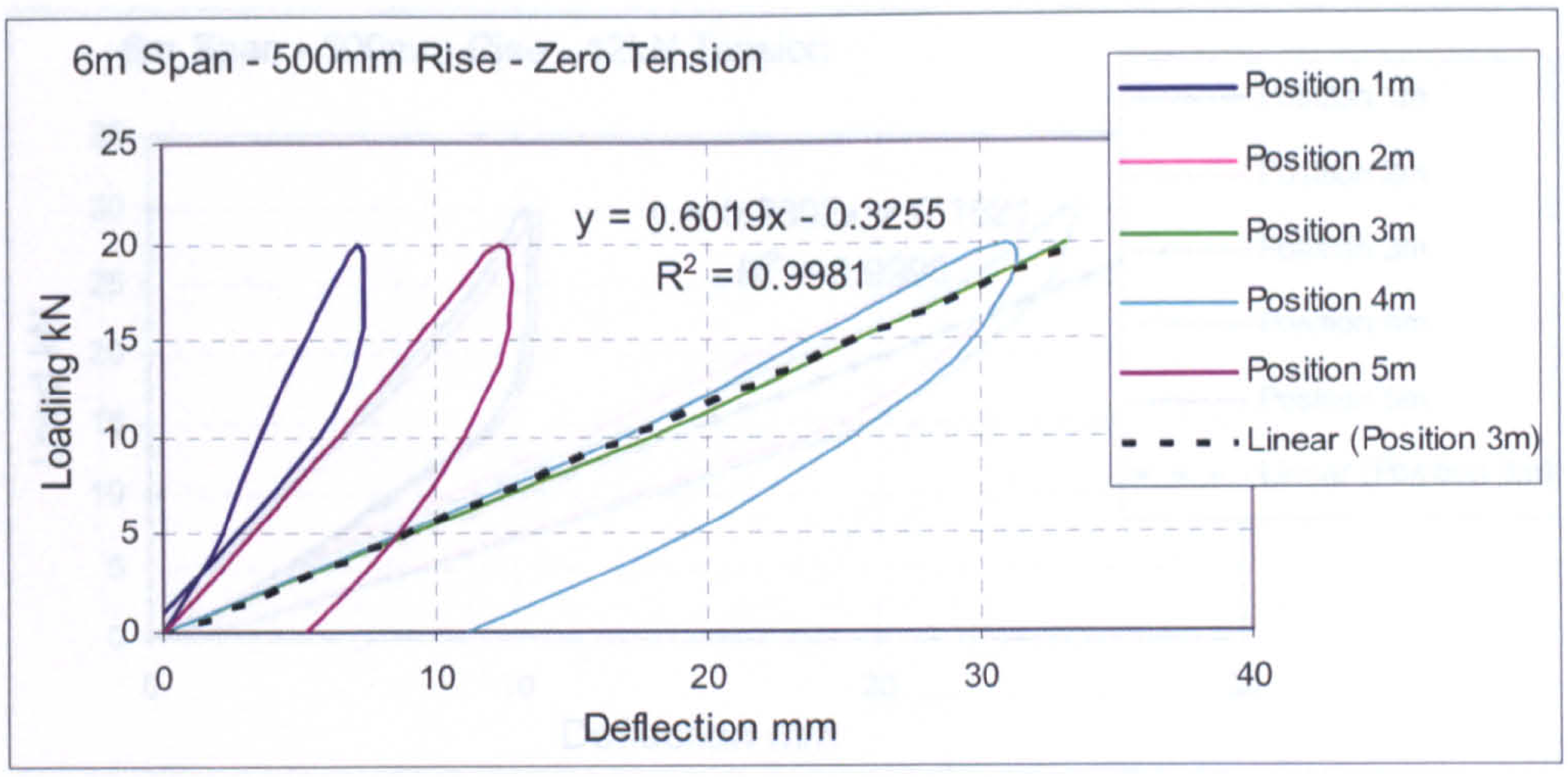


Fig 7.29a – 6m span – 500mm rise – zero tension

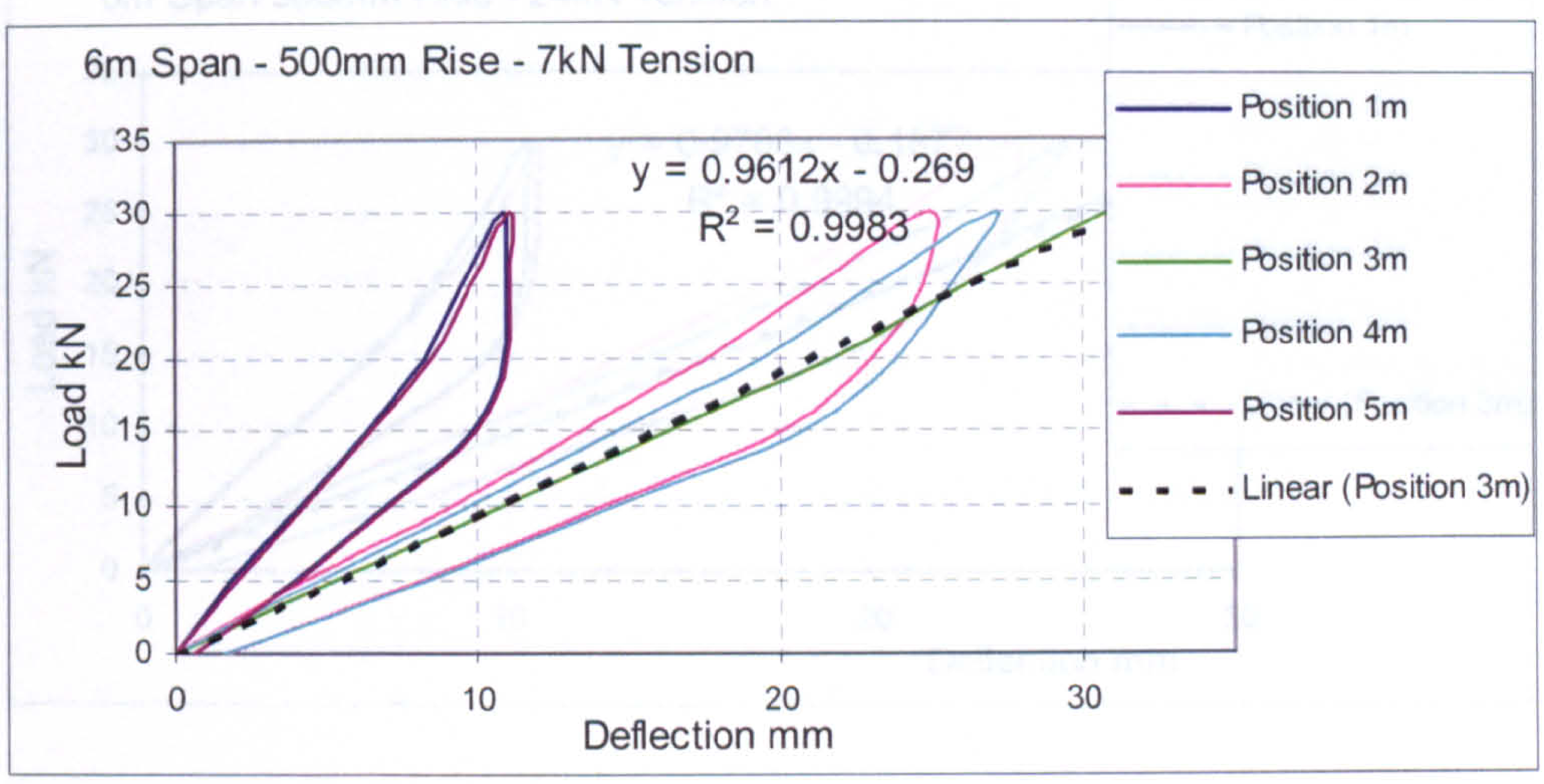


Fig 7.29b – 6m span – 500mm rise – 24kN tension

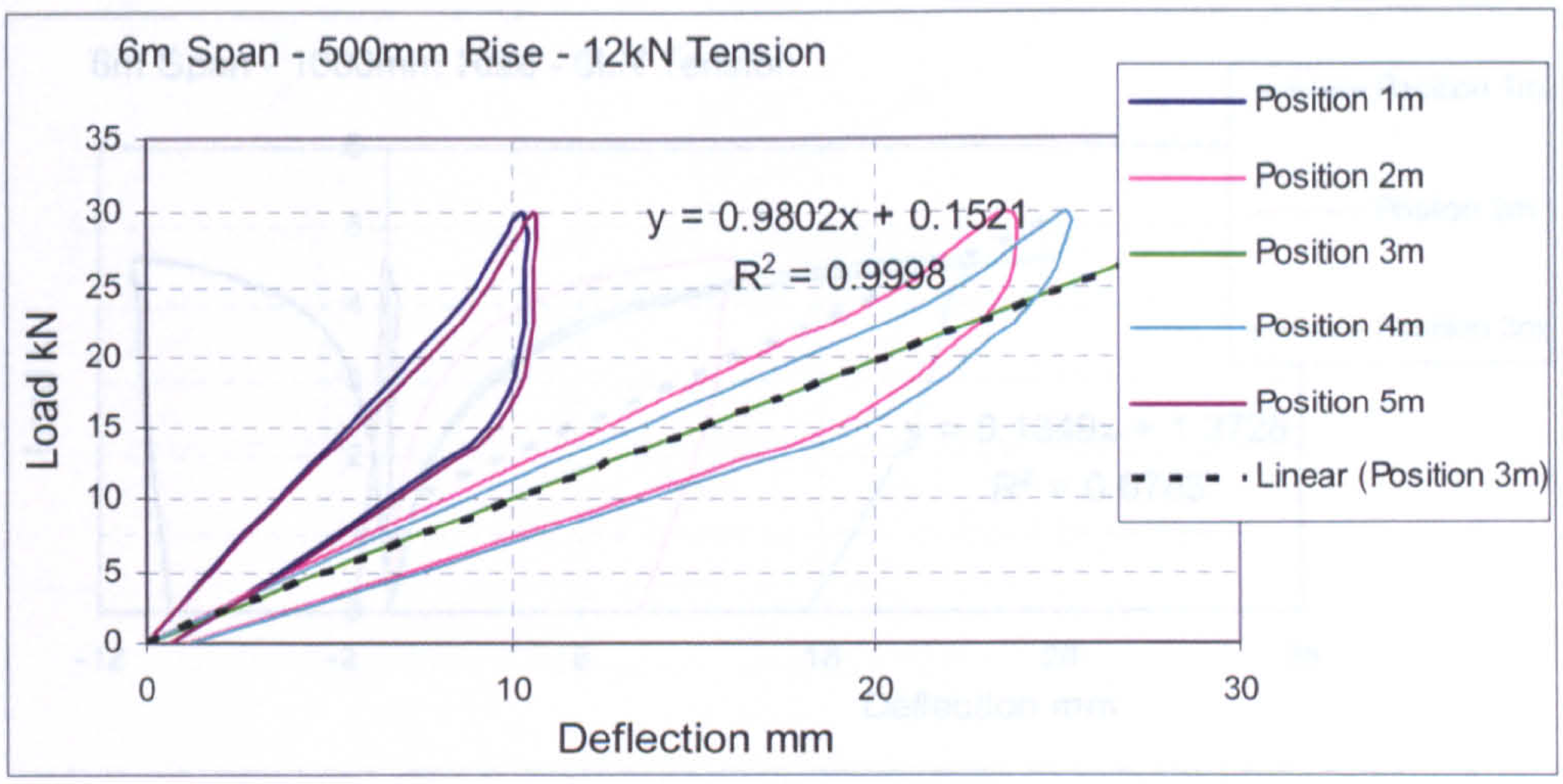


Fig 7.29c – 6m span – 500mm rise – 12kN tension

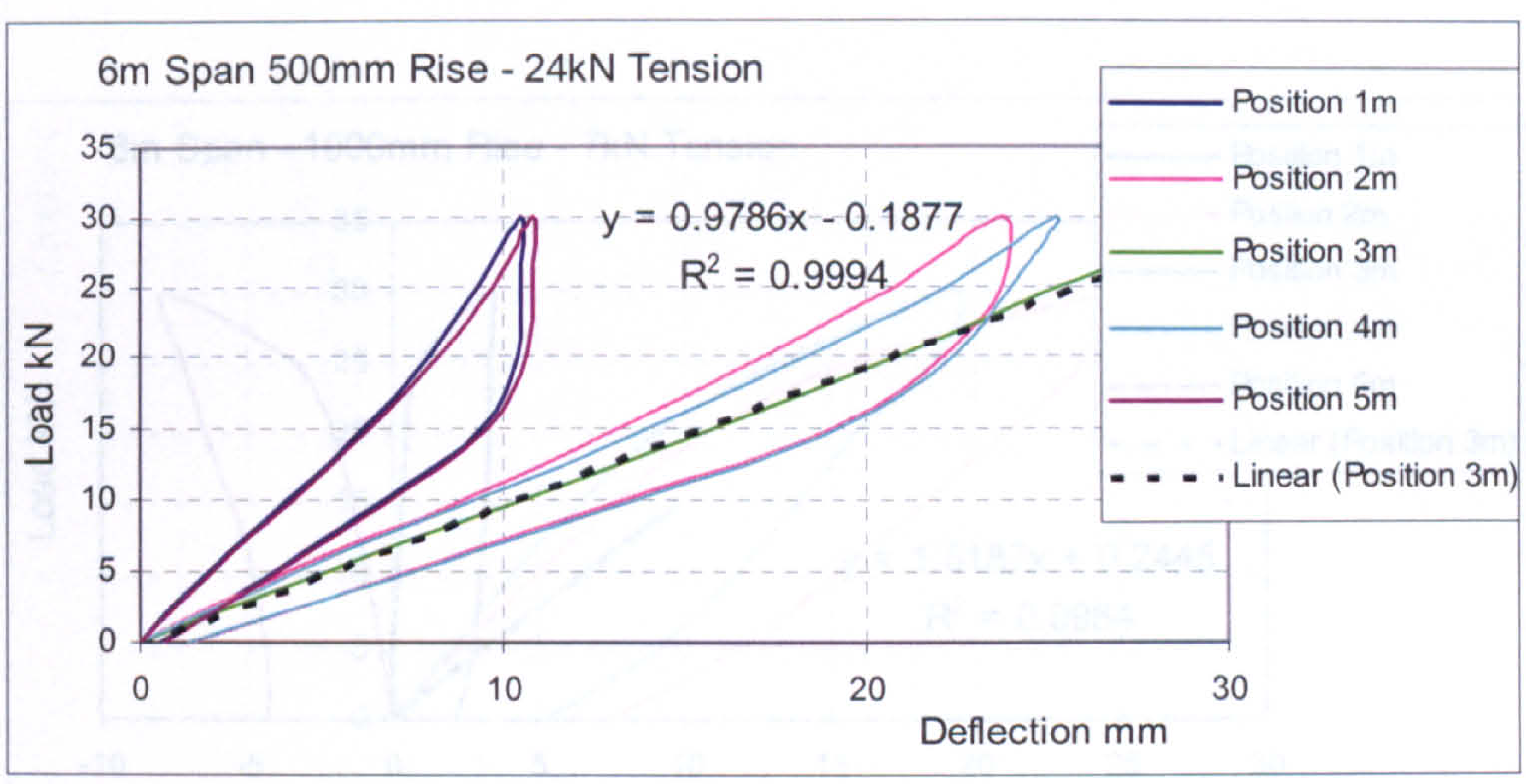


Fig 7.29d – 6m span – 500mm rise – 24kN tension

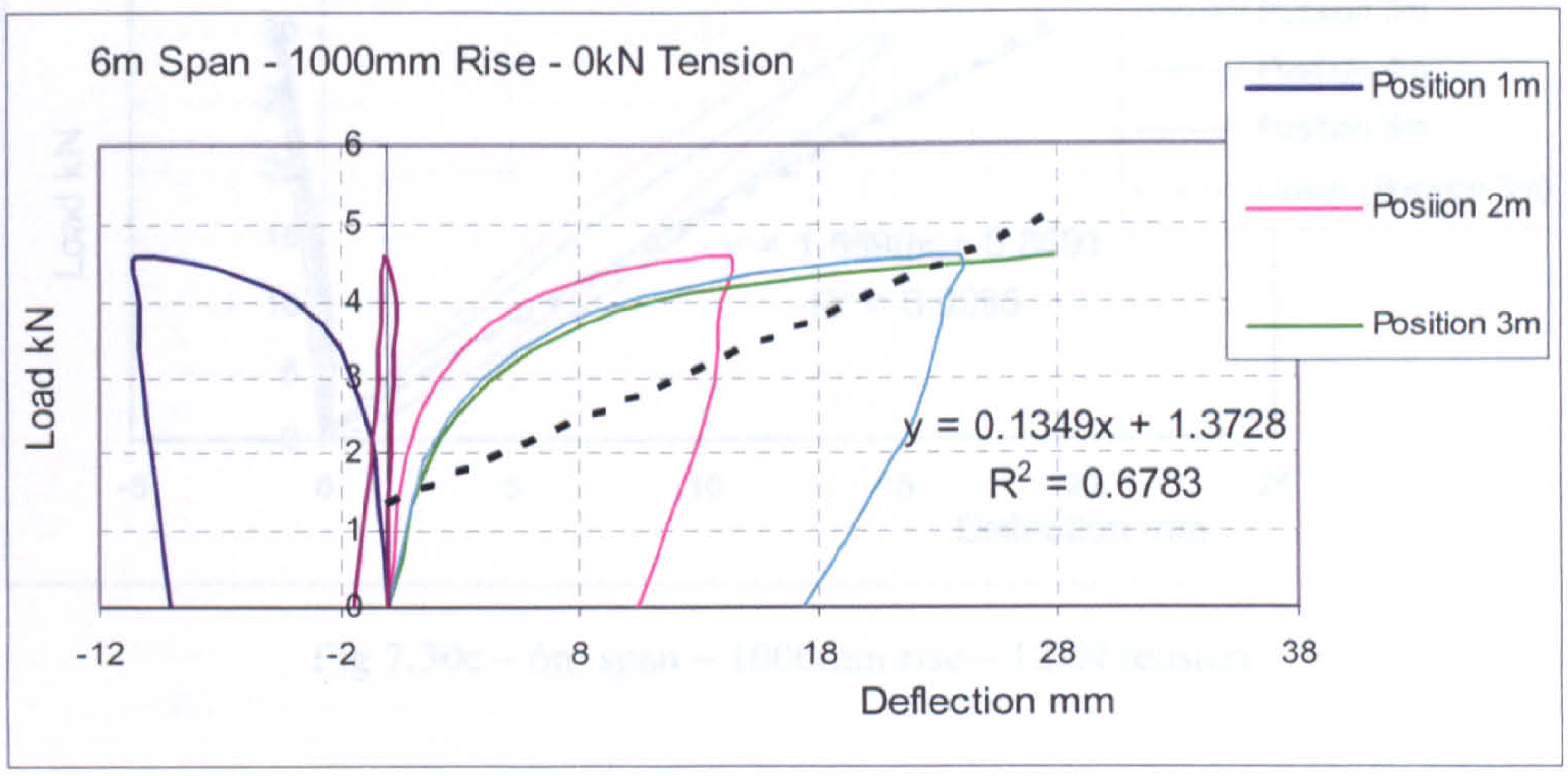


Fig 7.30a – 6m span – 1000mm rise – zero tension

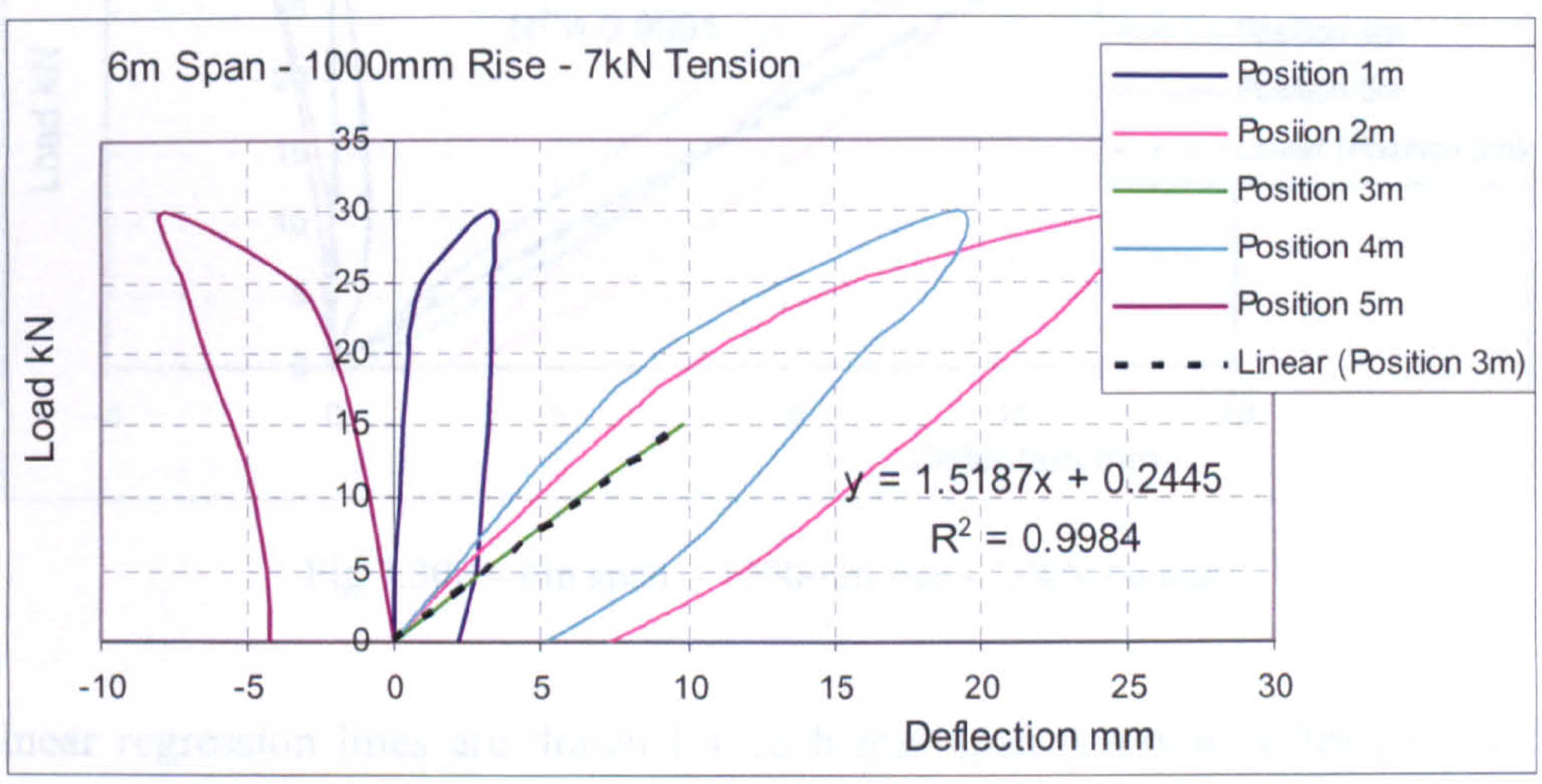


Fig 7.30b – 6m span – 1000mm rise – 7kN tension

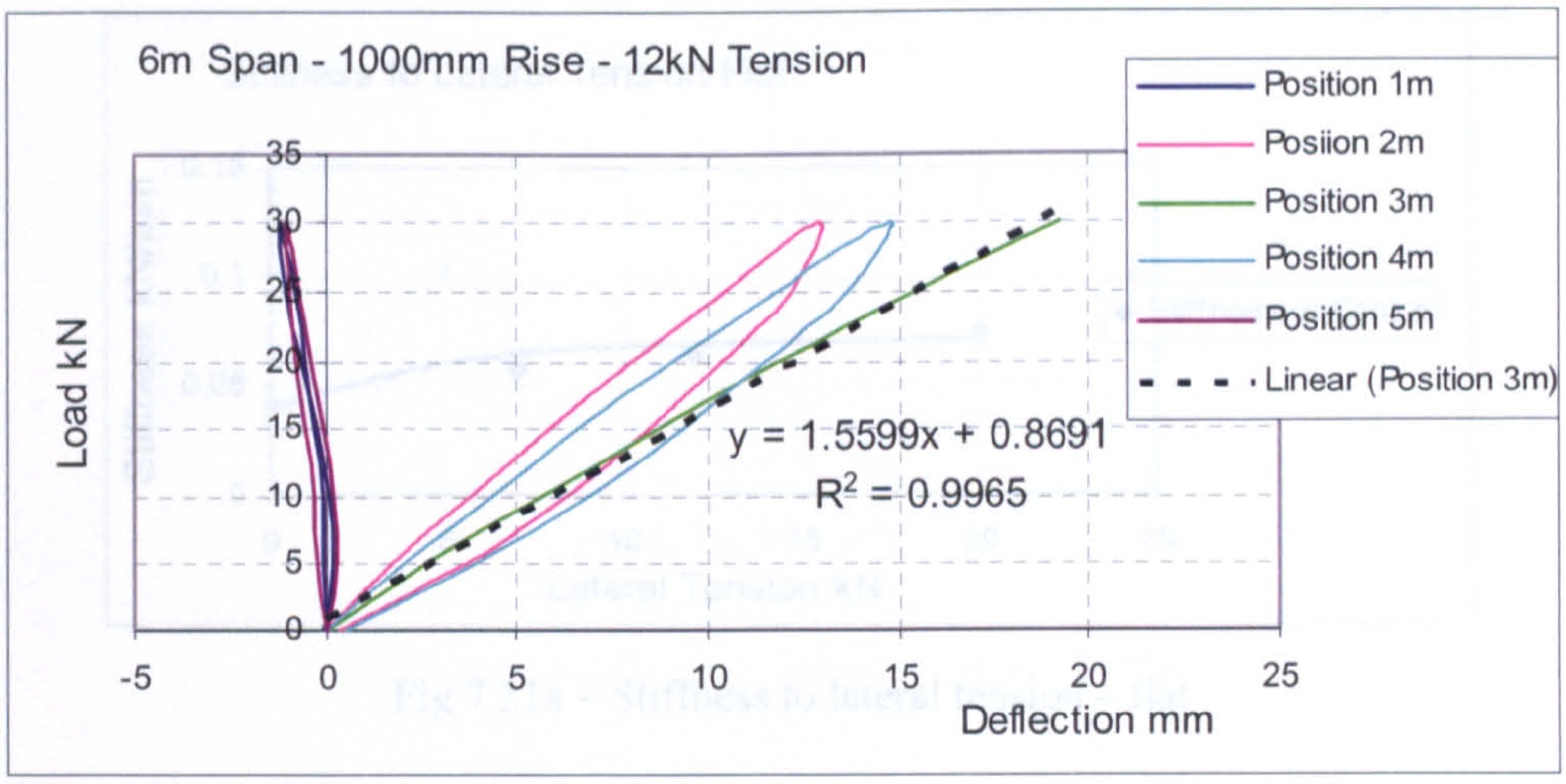


Fig 7.30c – 6m span – 1000mm rise – 12kN tension

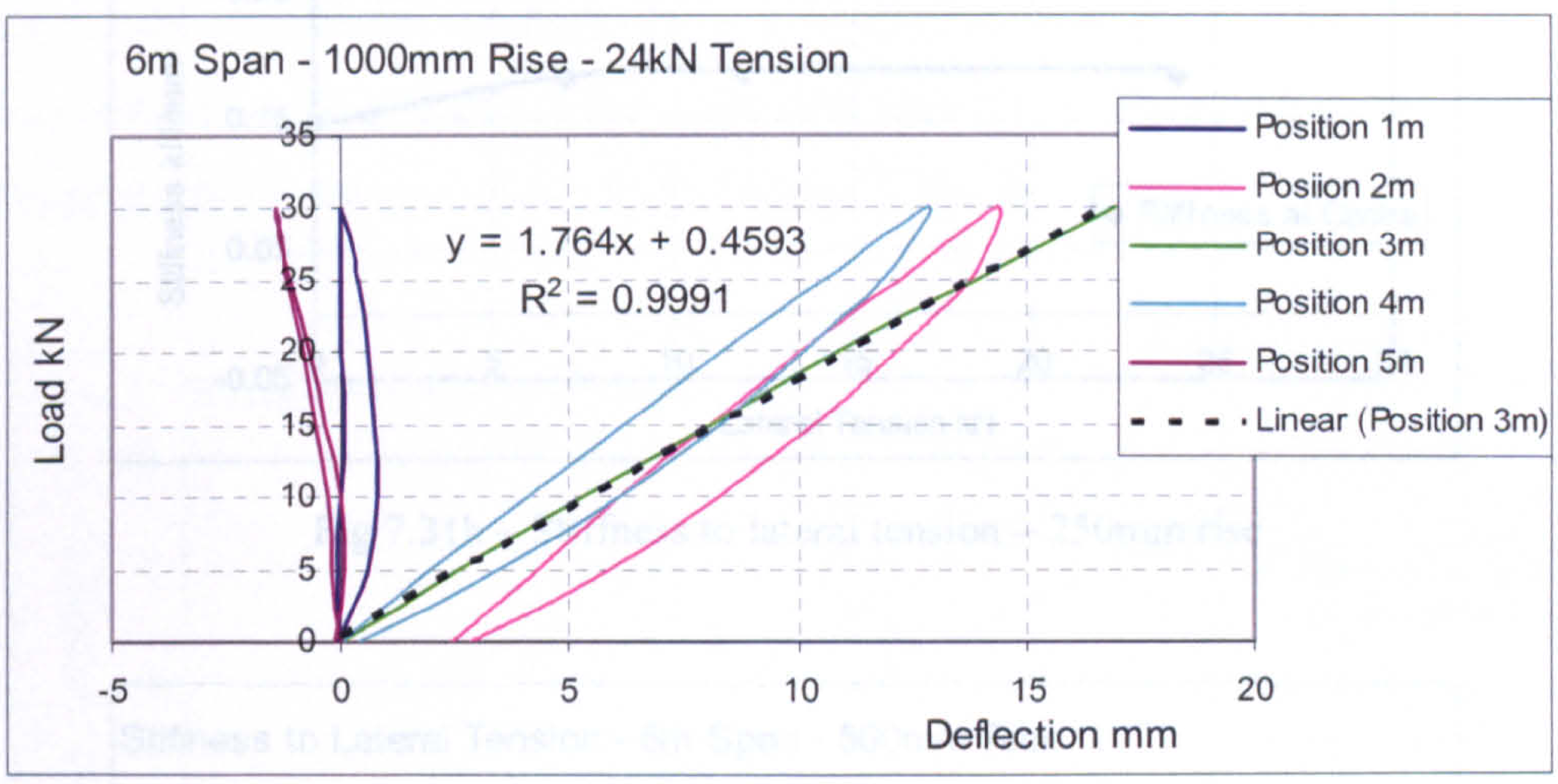


Fig 7.30d – 6m span – 1000mm rise – 24kN tension

The linear regression lines are drawn for each mid span position at 3m (heavy dotted line). The coefficient of the 'x' term is a stiffness measured in kN/mm. These, total deflection stiffnesses at mid span are plotted against the lateral tensions for each arch in Figures. 7.31a, b, c & d, to find the tension at which full stiffness is developed. The total deflection is the structural deflection plus the settlement deflection.

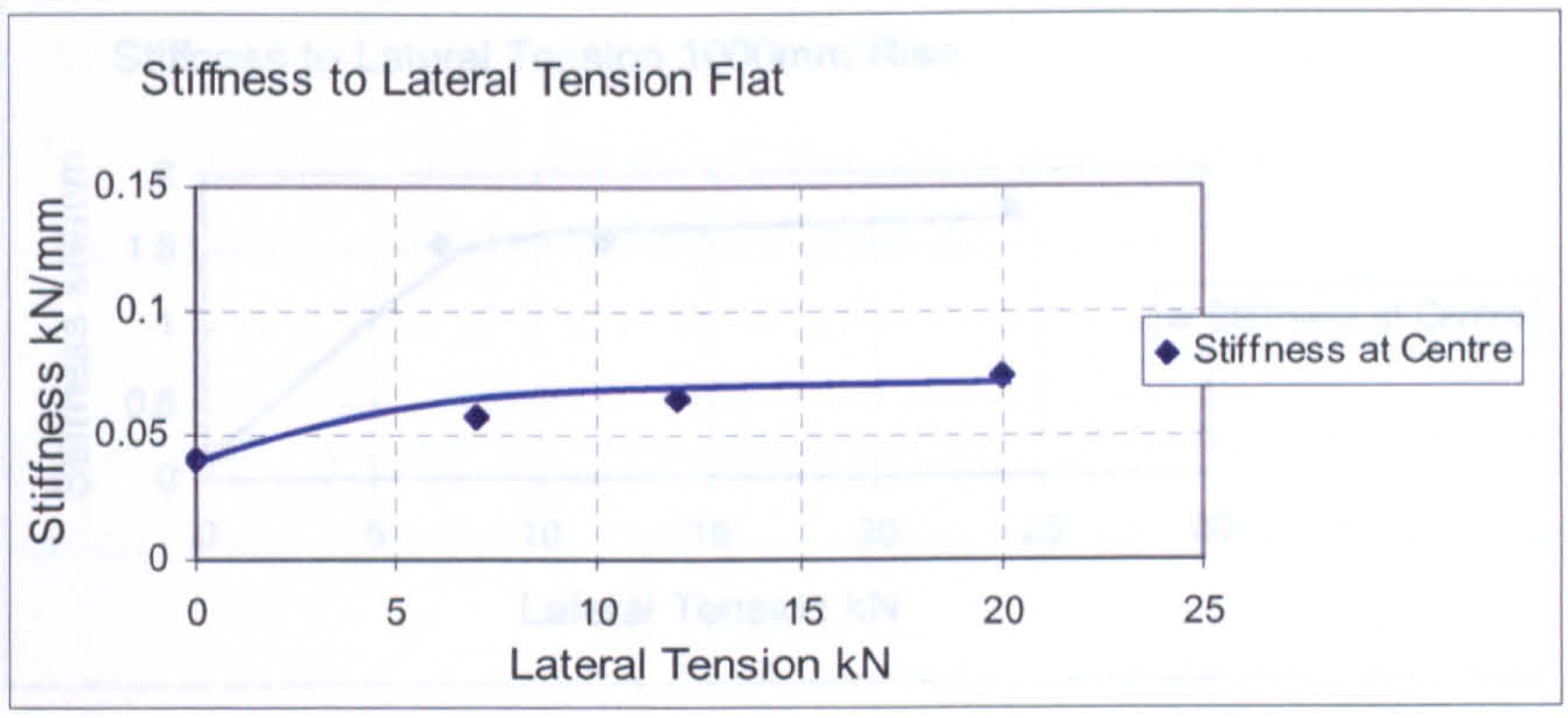


Fig 7.31a – Stiffness to lateral tension – flat

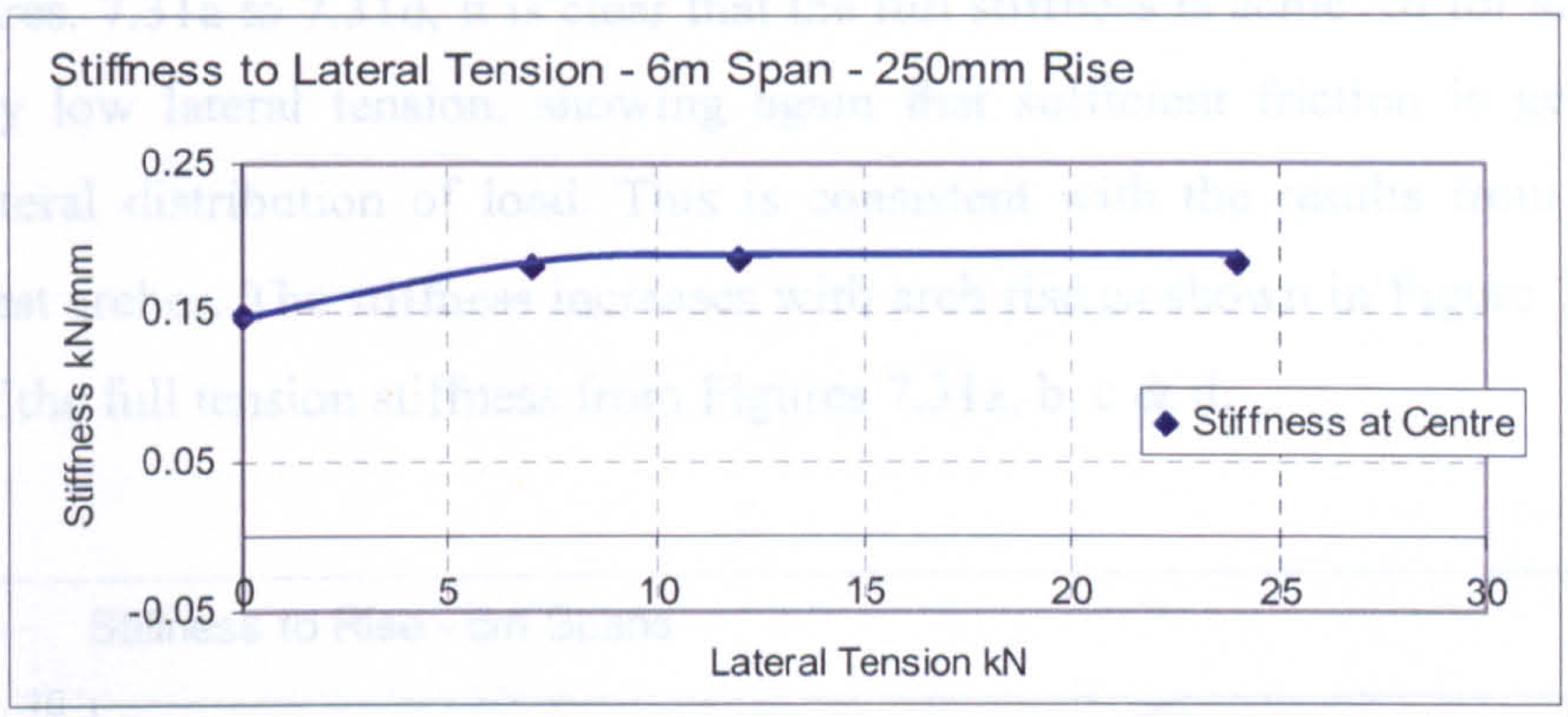


Fig 7.31b – Stiffness to lateral tension – 250mm rise

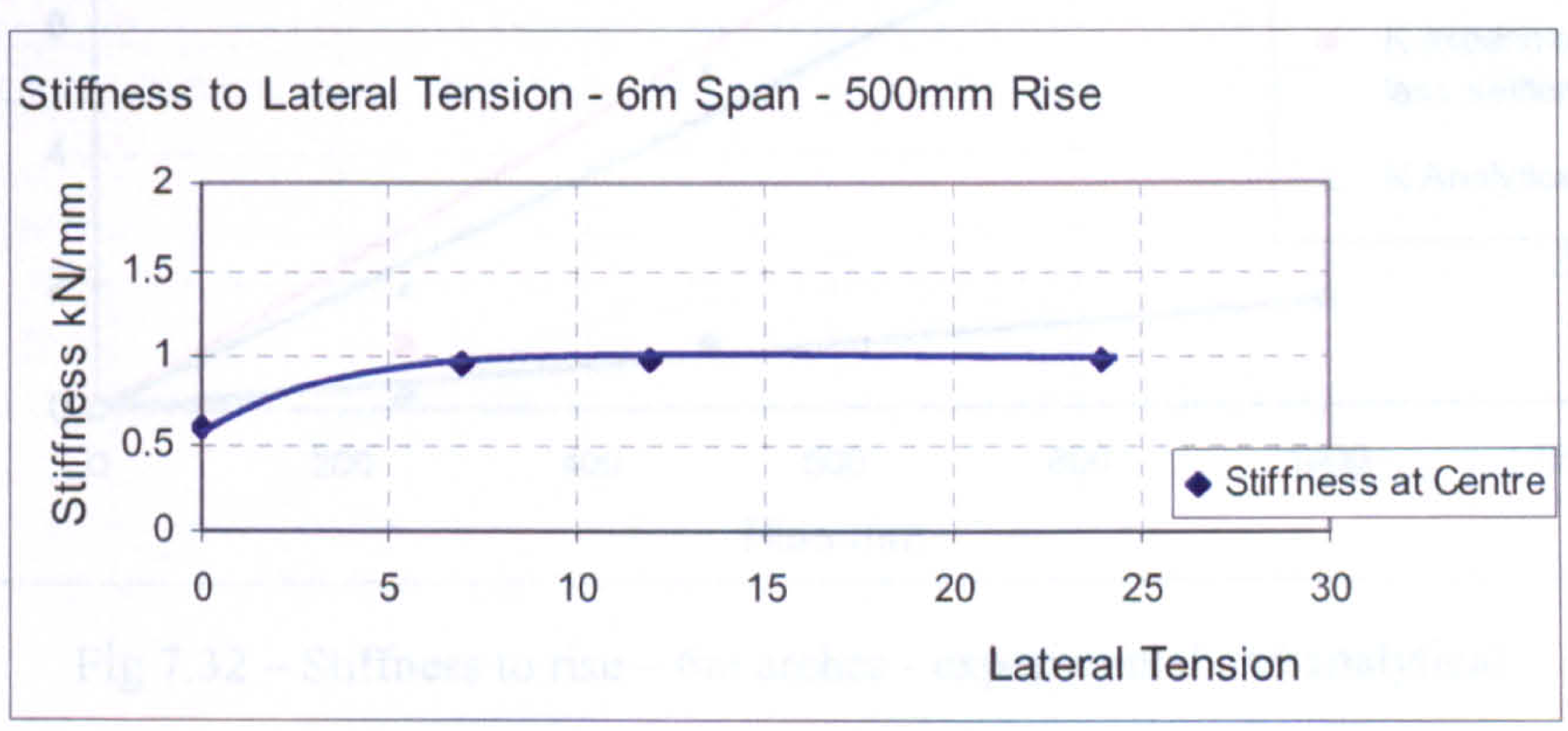


Fig 7.31c – Stiffness to lateral tension – 500mm rise

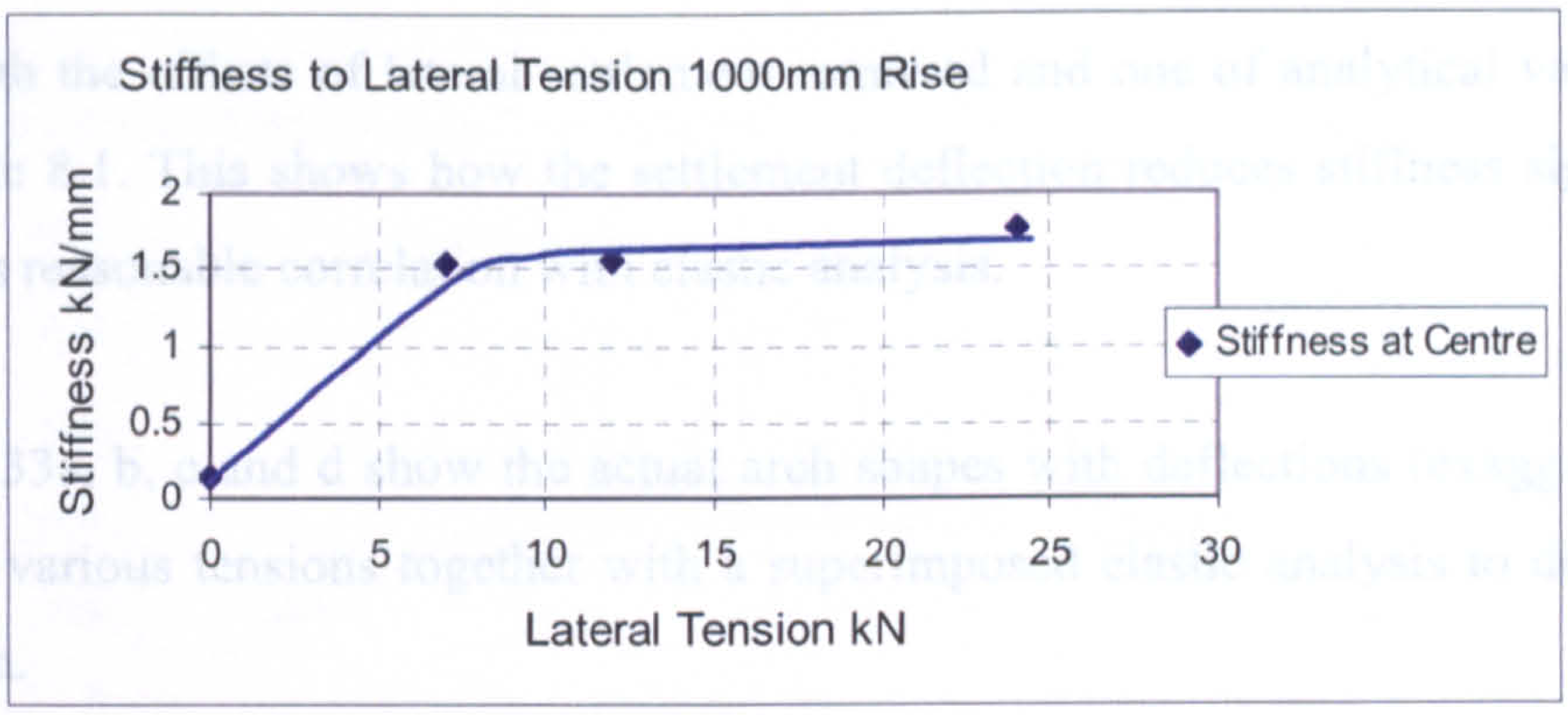


Fig 7.31d – Stiffness to lateral tension – 1000mm

From Figures. 7.31a to 7.31d, it is clear that the full stiffness is achieved for all arches at a relatively low lateral tension, showing again that sufficient friction is generated to provide lateral distribution of load. This is consistent with the results from all of the previous test arches. The stiffness increases with arch rise as shown in Figure 7.32 which is a plot of the full tension stiffness from Figures 7.31a, b, c & d.

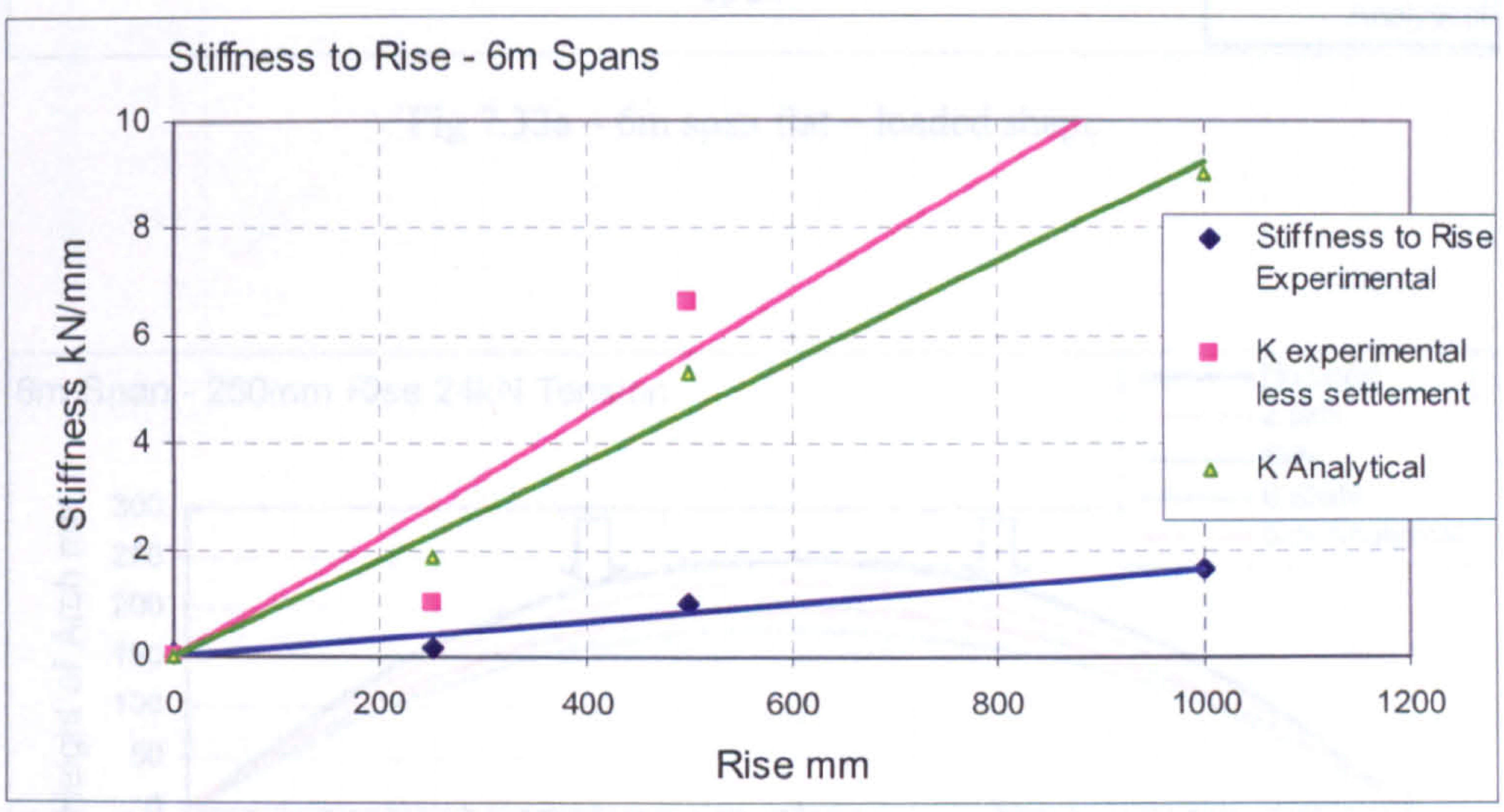


Fig 7.32 – Stiffness to rise – 6m arches - experimental and analytical

Figure 7.32 shows increasing stiffness from flat spans up to a span to rise ratio of one to six. It has been shown for the 2.1m spans, and will be confirmed in Chapter 8 that there is an optimum span to rise at one to five. Figure 7.32 includes plots of the experimental results with the effects of lateral settlement removed and one of analytical values taken from Table 8.1. This shows how the settlement deflection reduces stiffness significantly and shows reasonable correlation with elastic analysis.

Figures 7.33a, b, c and d show the actual arch shapes with deflections (exaggerated five times), at various tensions together with a superimposed elastic analysis to demonstrate correlation.

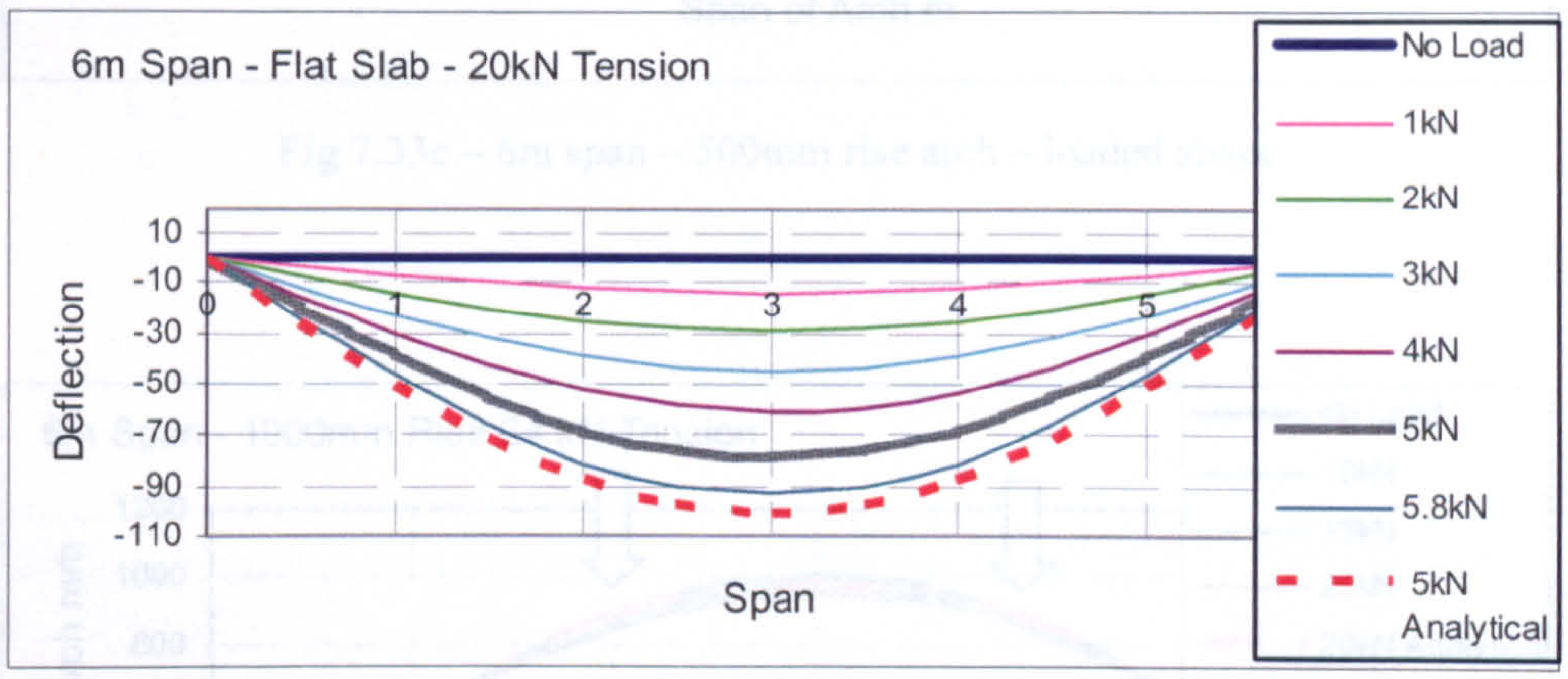


Fig 7.33a – 6m span flat – loaded shape

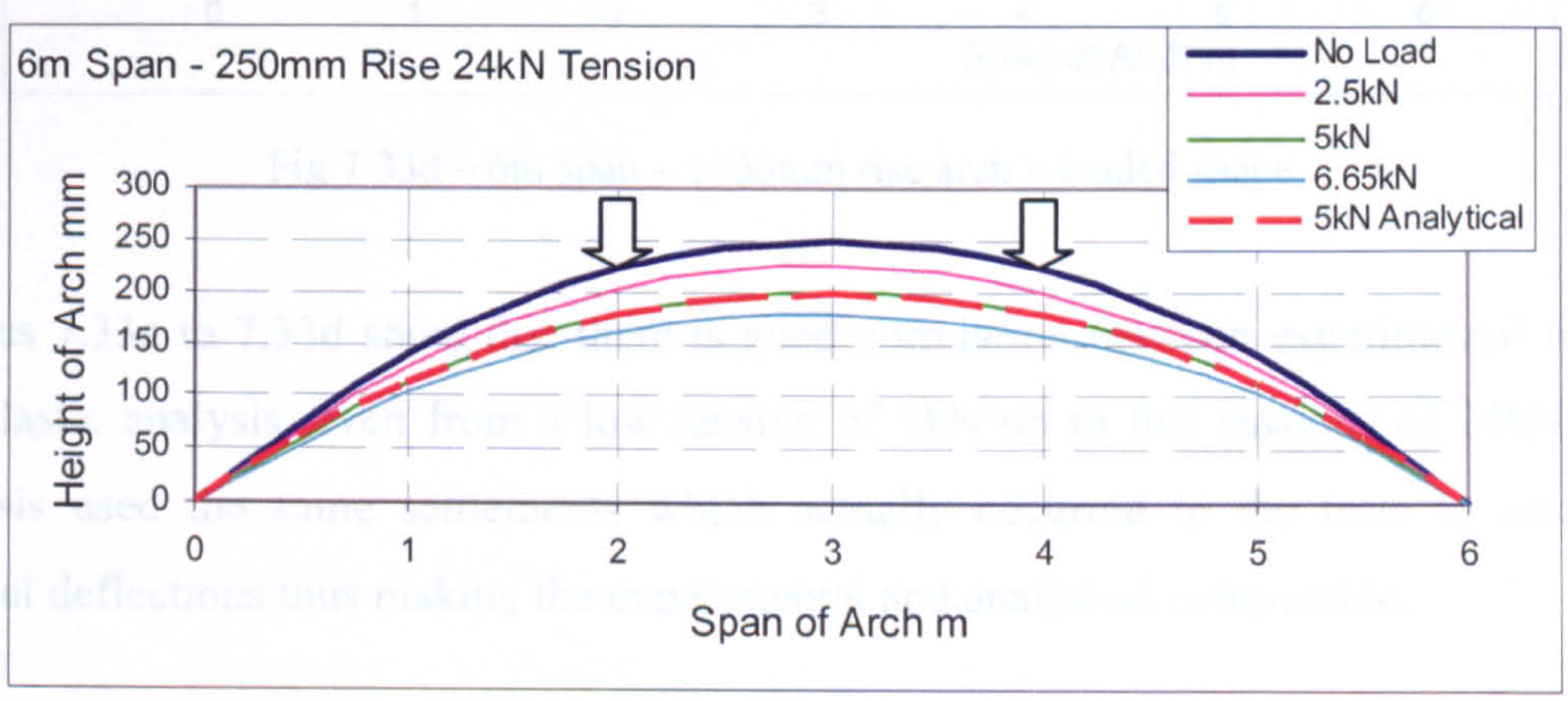


Fig 7.33b – 6m span – 250mm rise arch – loaded shape

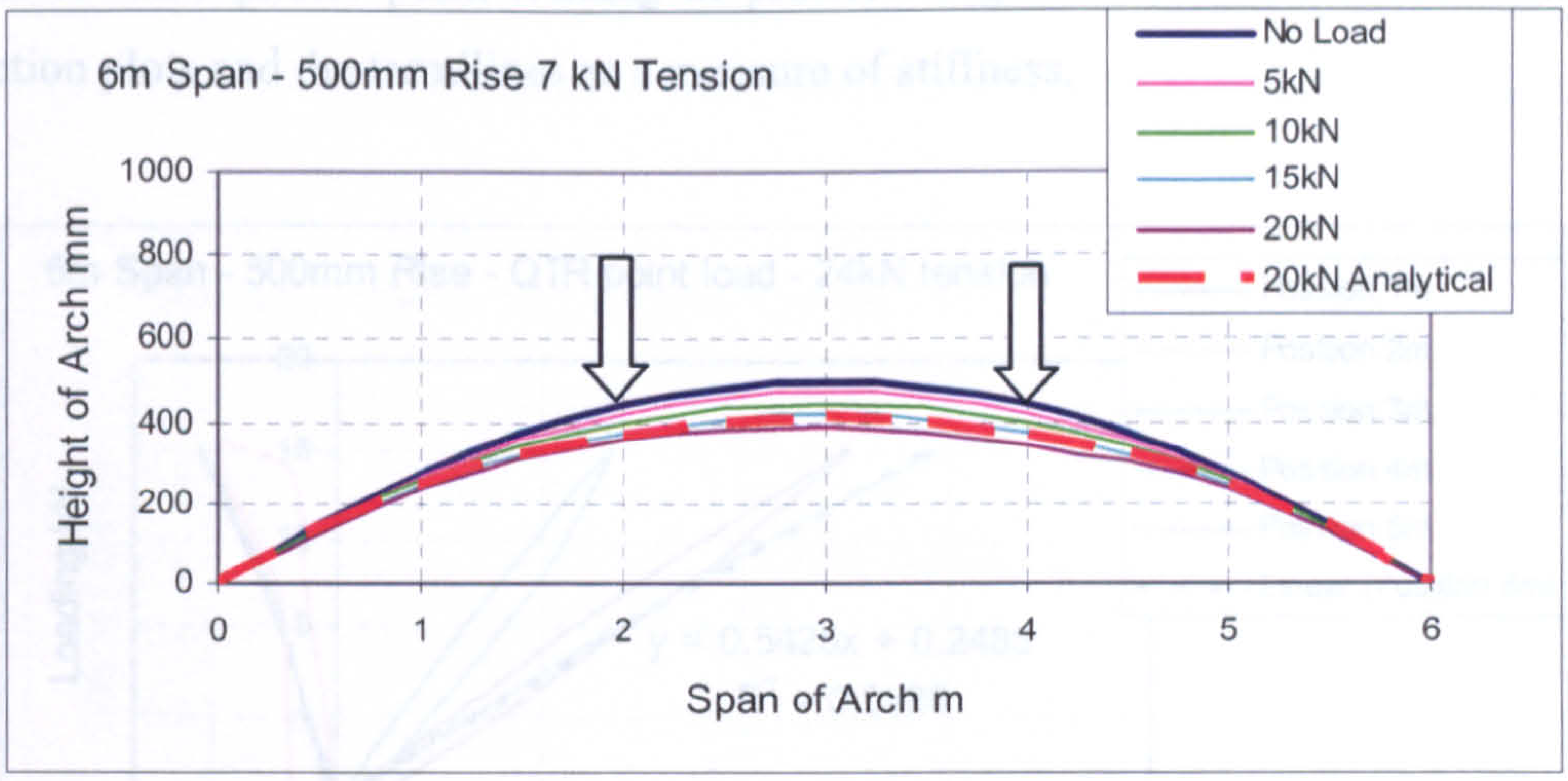


Fig 7.33c – 6m span – 500mm rise arch – loaded shape

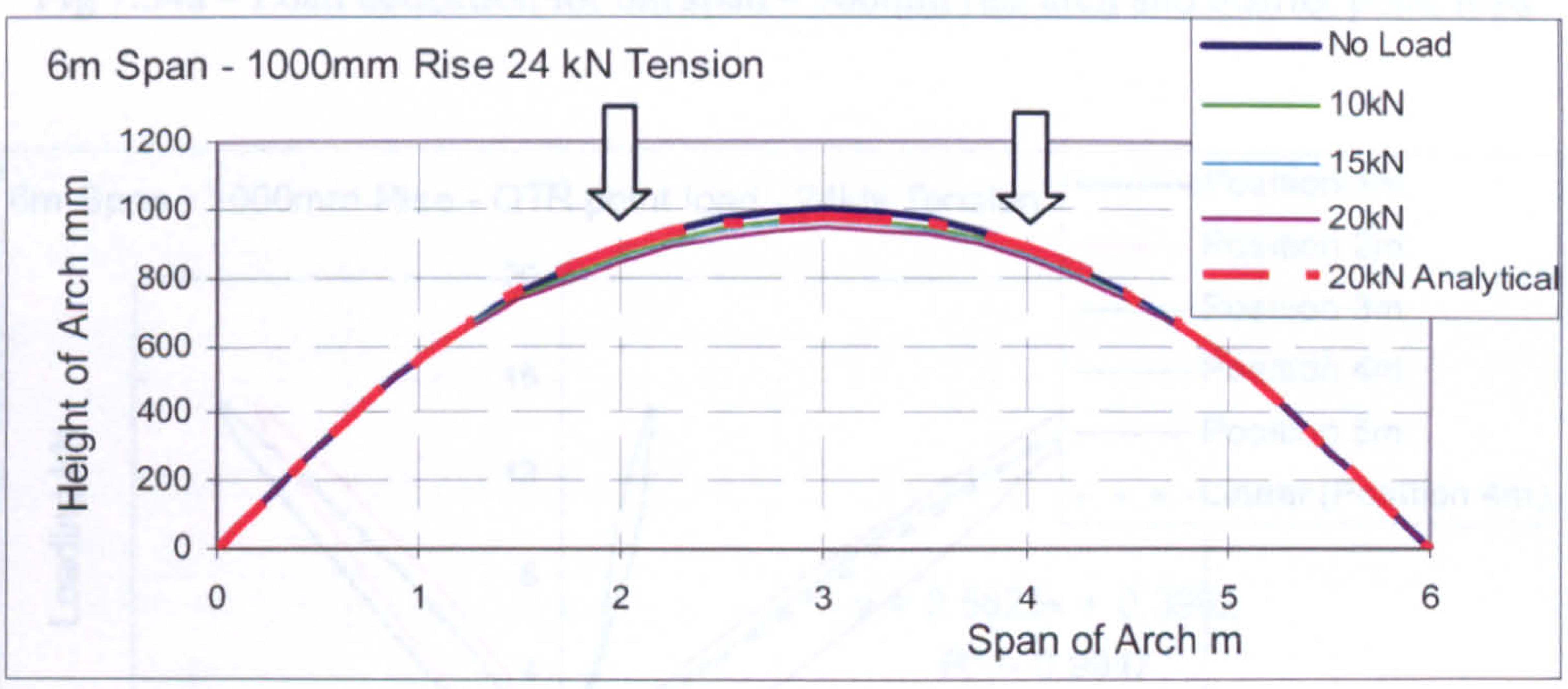


Fig 7.33d – 6m span – 1000mm rise arch – loaded shape

Figures 7.33a to 7.33d show that there is good correlation between experimental results and elastic analysis, even from a low tension of 7kN up to full tensions of 24kN. The analysis used the same settlements which actually occurred in the tests to calculate vertical deflections thus making the experimental and analytical comparable.

As a concluding test the 500mm and 1000mm rise arches were loaded at their quarter points. As this is the weakest point the stiffness will become the design stiffness if the arch is to take quarter point loading in practice. Figures 7.34a and b show the load deflection plots and the trendlines as a measure of stiffness.

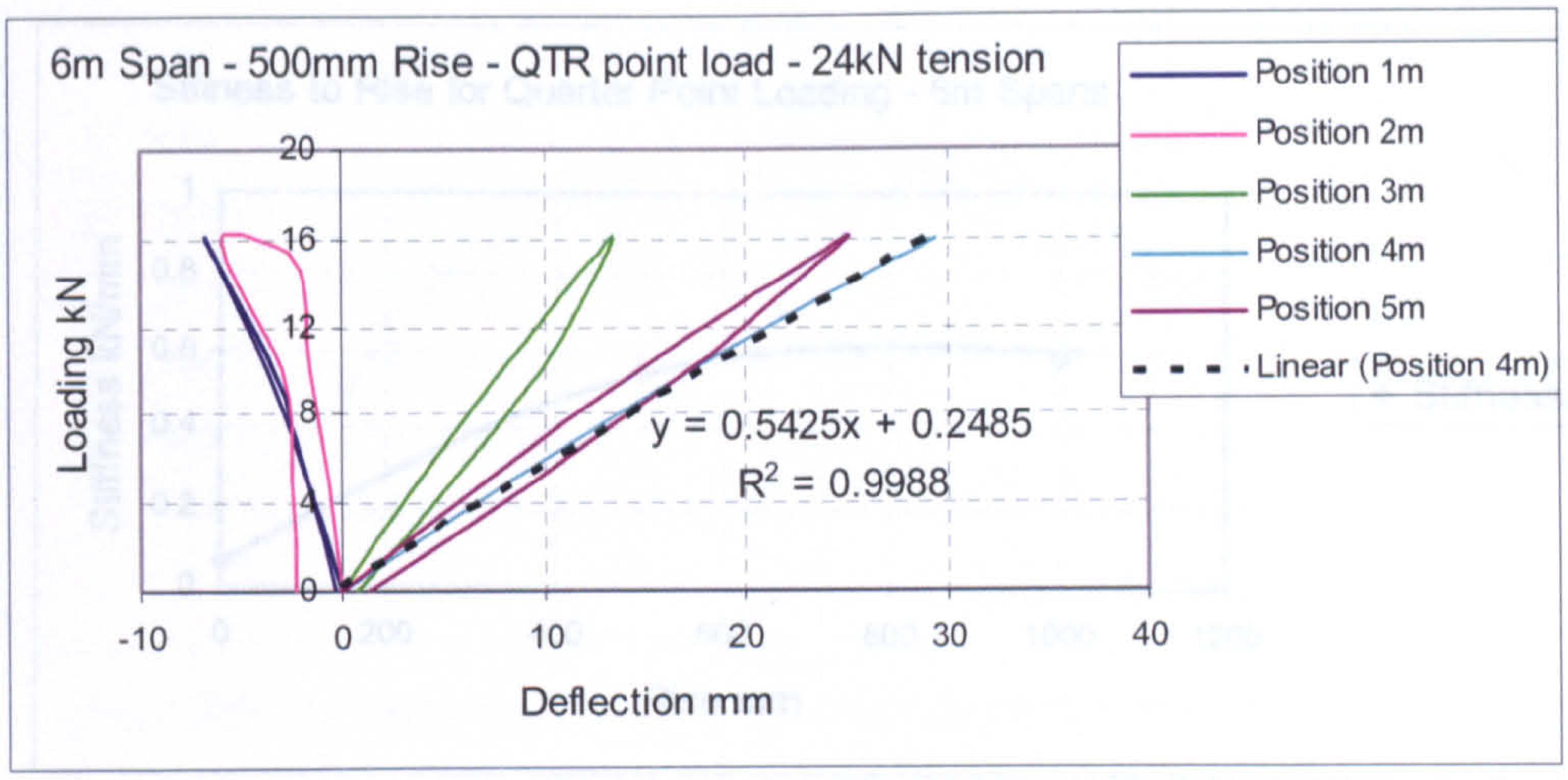


Fig 7.34a – Load deflection for 6m span – 500mm rise arch and quarter point load

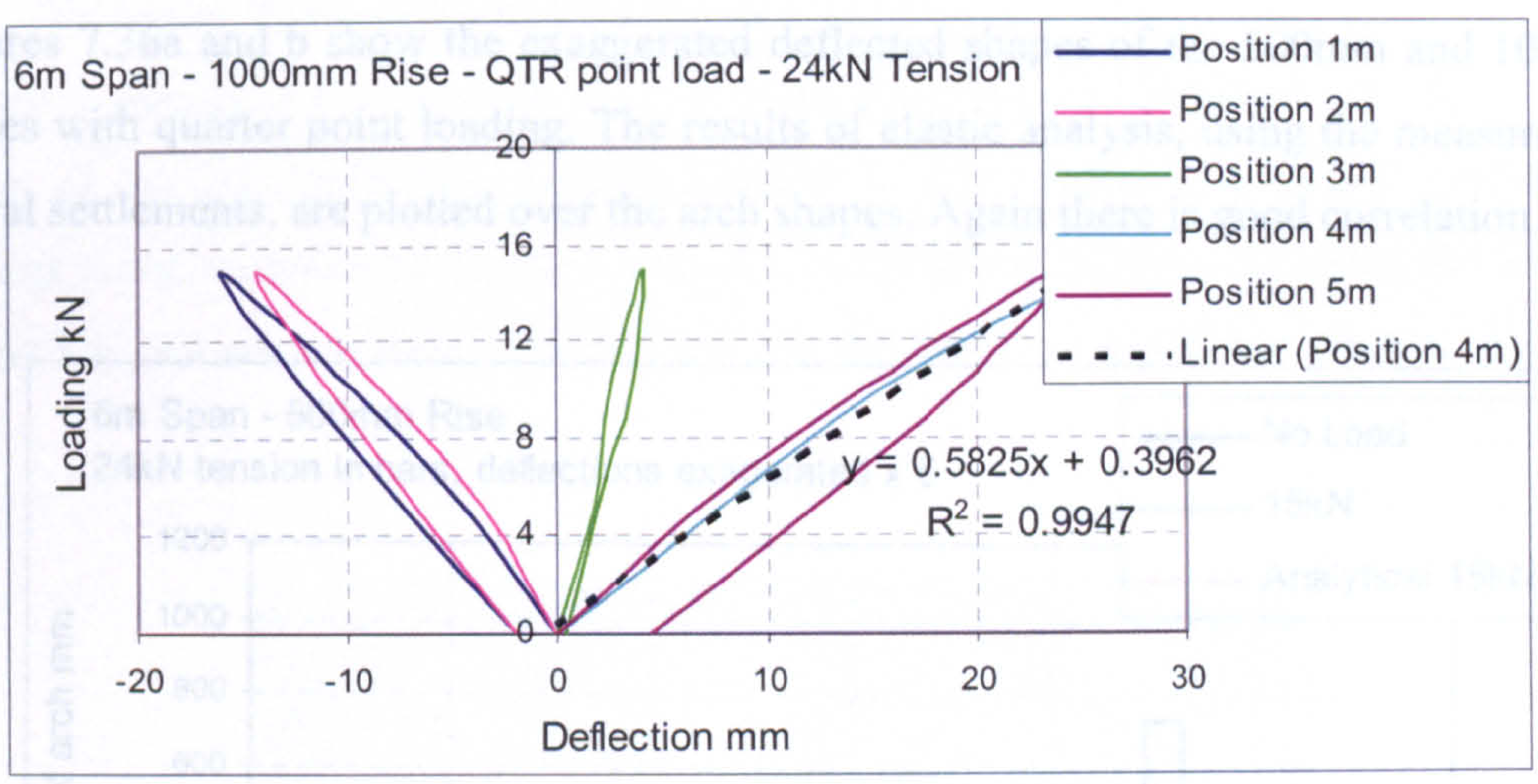


Fig 7.34b – Load deflection for 6m span – 1000mm rise arch and quarter point load

The stiffnesses from Figures 7.34a and b were plotted against the rises using the flat span stiffness from the four point load tests as this would not change for quarter point load. This is shown in Figure 7.35 and in this case the increase in stiffness for the higher rise is not as great because of the deflected shape the arch takes with asymmetric loading. The quarter point load introduces a high proportion of bending whereas the symmetrical four point loading induces predominantly compressive stresses resulting in less deflection.

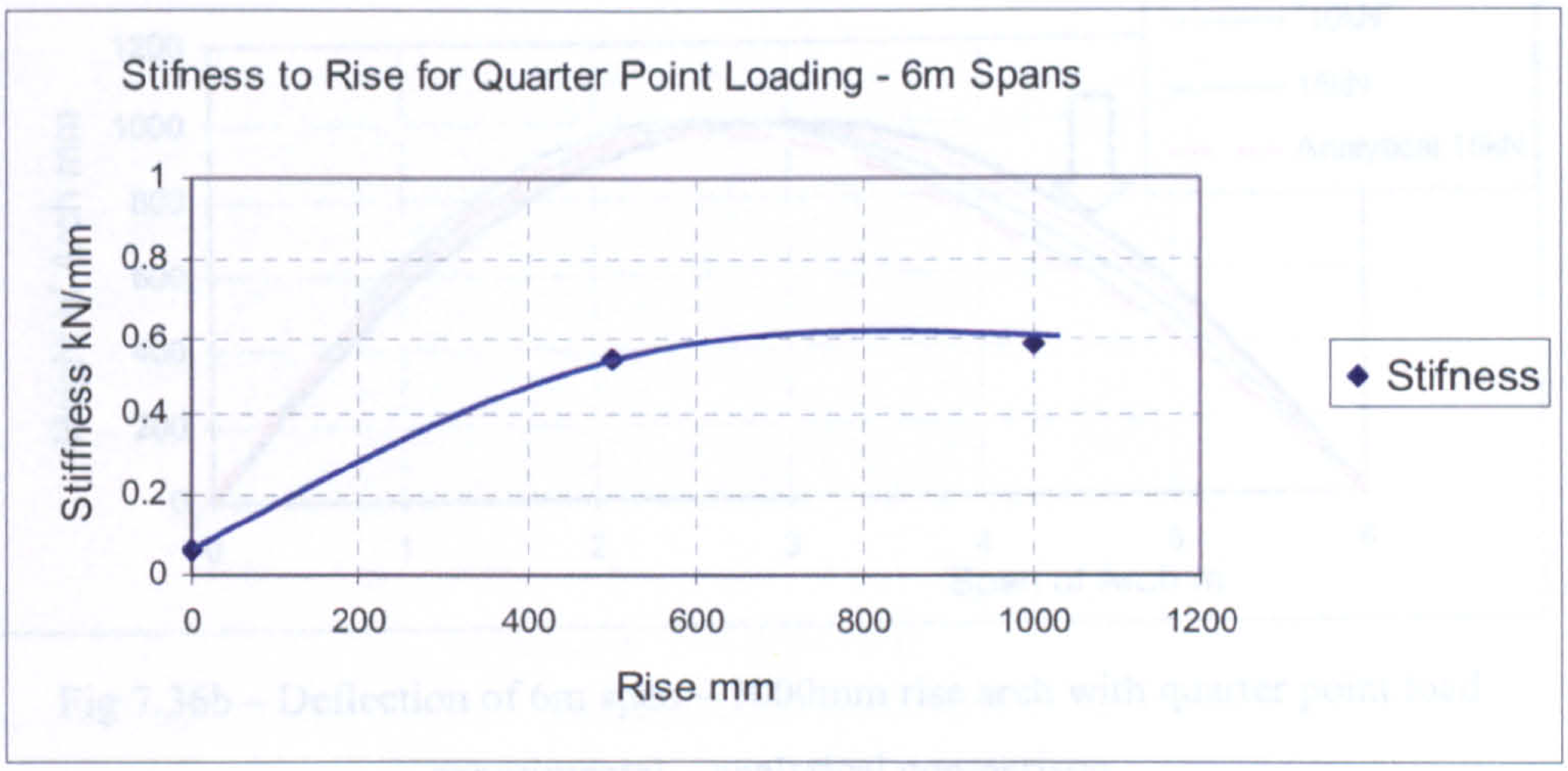


Fig 7.35 – Stiffness to rise for quarter point loading

Figures 7.36a and b show the exaggerated deflected shapes of the 500mm and 1000mm arches with quarter point loading. The results of elastic analysis, using the measured test lateral settlements, are plotted over the arch shapes. Again there is good correlation.

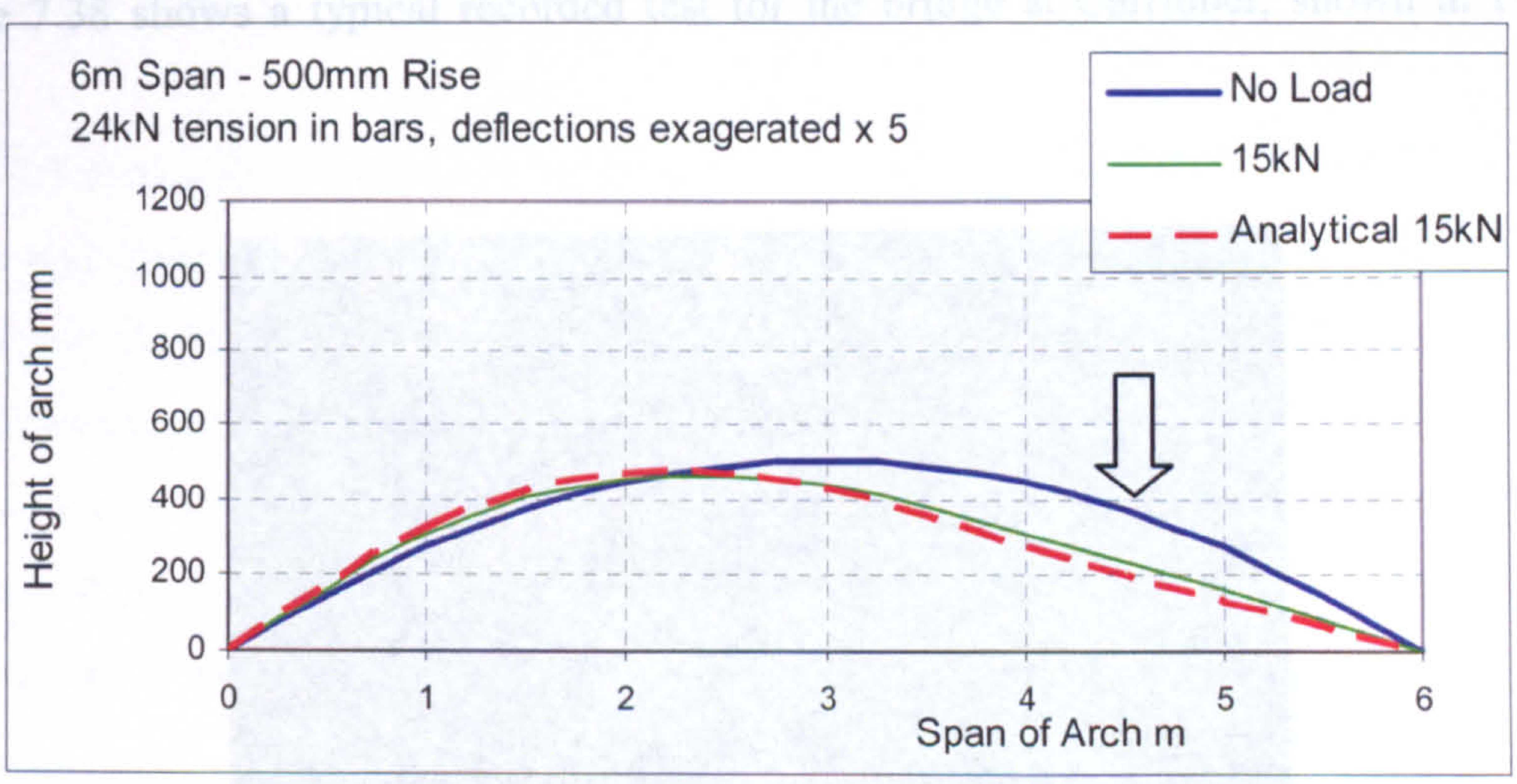


Fig 7.36a – Deflection of 6m span – 500mm rise arch with quarter point load experimental – analytical comparison

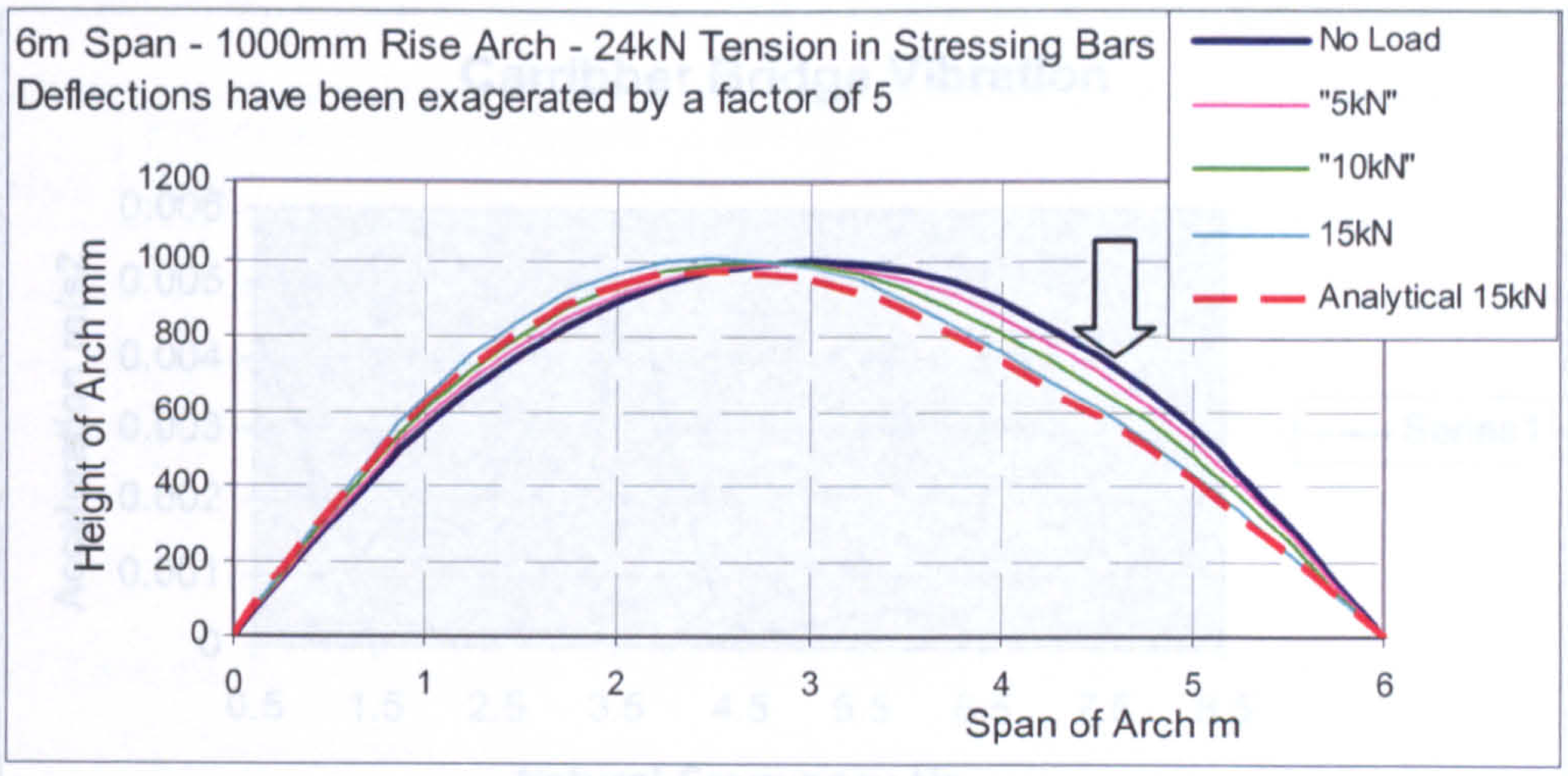


Fig 7.36b – Deflection of 6m span – 1000mm rise arch with quarter point load experimental – analytical comparison

7.7 Dynamic Tests on Field Structures

The three measurement methods of FNF were described in Chapter 6 and the detailed testing of the 20m span was described earlier in this chapter. To further confirm the findings some commercial bridges, recently constructed, were tested using the RT440. Figure 7.38 shows a typical recorded test for the bridge at Carribber, shown in Figure 7.37.

Fig 7.37 – Carribber bridge

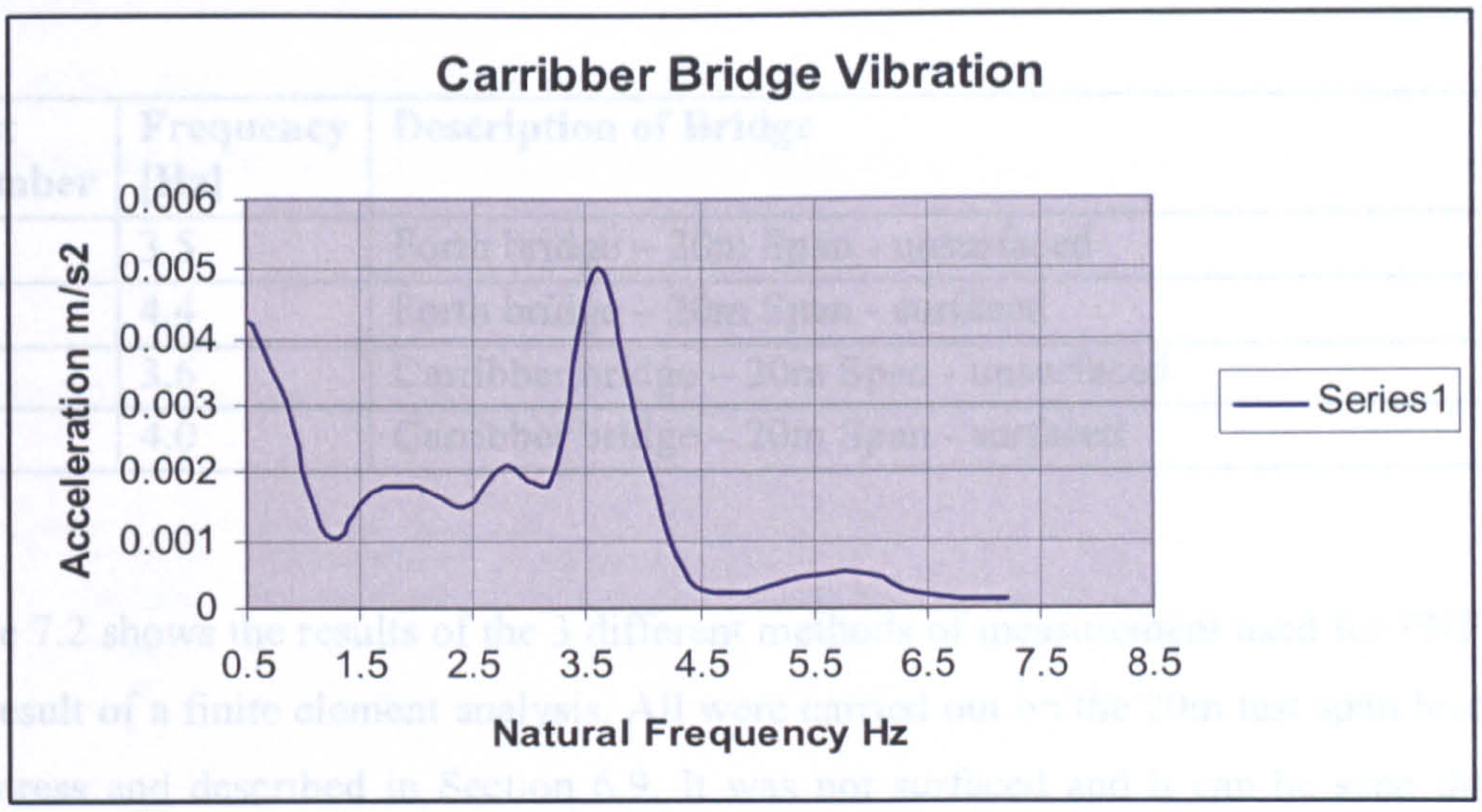


Fig 7.38 - Output from vibration Carribber bridge

Figure 7.38 shows the Natural Frequency of the 20m span Carribber bridge to be 3.6Hz. There is a rogue peak at the axis of approximately 0.5Hz. This is the decay of the Fourier Series which is applied to the vibration meter output, to organize it into actual meaningful peaks. The maximum acceleration is .005m/s^2, which is very much less than $0.5\sqrt{f_0}$ given by Jiri Strasky [73] as the comfortable maximum for pedestrians.

Table 7.1 shows the field results taken up to this time. It is significant that both the Forth bridge and Carribber bridge FNFs increased after the bitumen topping was added. This shows that the topping added stiffness by bonding to the deck. If it had not bonded it would have added dead weight and decreased the FNF. All of the completed bridges will be tested as a continuation of this work to build up a database and to check that they maintain their stiffness. A decrease in value through the life of the bridge would show that the lateral tension had relaxed or the topping had delaminated or structural decay had occurred.

Table 7.1 – Dynamic Field Test Results

Test Number	Frequency [Hz]	Description of Bridge
1	3.5	Forth bridge – 20m Span - unsurfaced
2	4.4	Forth bridge – 20m Span - surfaced
3	3.6	Carribber bridge – 20m Span - unsurfaced
4	4.0	Carribber bridge – 20m Span - surfaced

Table 7.2 shows the results of the 3 different methods of measurement used for FNF with the result of a finite element analysis. All were carried out on the 20m test span bridge at Glentress and described in Section 6.9. It was not surfaced and it can be seen that the values agree with the Forth and Achray bridges of the same span, Table 7.1.

Table 7.2 - Dynamic test results on a 20m span unsurfaced bridge using three different instruments

Test Number	Frequency [Hz]	Equipment and Excitation
1	3.54	ARTeMIS test planner and modal analyses software Excitation by walking
2	3.6	dual spectrum analyser Hammer impact excitation
3	3.5	Handheld vibration analyser RT440 50kg sandbag dropped on deck as excitation
4	4	Finite Element analysis

7.8 Materials Testing Results

The data from eight test samples from the 2.1m span laboratory arches and ten from the 6m spans were averaged and used to calculate the values given in Table 7.3. The calculations were carried out in accordance with the equations given in BS EN 408:1995 [81].

Table 7.3 – Materials properties of timber samples from 2.1m and 6m laboratory arches.

Span of Bridge sample m	Modulus of elasticity Bending N/mm^2	Moisture Content %age	E Compression 0^0 N/mm^2	E Compression 45^0 N/mm^2	E Compression 90^0 N/mm^2
2.1	4590				
6	12642	10.76	2297	680	310

The values for Modulus of Elasticity for bending are the most significant. The timbers used for the 2.1m span bridges were poor quality but the bridges still performed well showing the value of taking load in compression. The 6m span bridges were built from better timber as shows in the result which is 40% greater than the 8800N/mm^2 value used in analysis comparisons because C16 was assessed by visual grading. The moisture content is very low because the timber had been in the laboratory for some time. The compression values show the significantly greater capacity parallel to the grain which again shows the value of arches which take most of the load as a compression parallel to the grain.

7.9 Conclusions from Experimental Results

One of the objectives of this PhD research was to study and quantify the behaviour of stress laminated solid timber arches for bridge decks and develop a predictive empirical model that would represent the structural behaviour within the range of expected loadings and design conditions. The above work, to some extent, achieved this objective and produced a number of useful outcomes.

From the beginning, the focus was on finding the critical parameters which would lead to the optimum design for particular requirements. This study emerged originally from a commercial application for pedestrian use. It was therefore biased towards a shape which would permit easy access over the deck. This is why the first arch to be tested had a span to rise ratio of 1 to 12.

Flat arches of a similar profile had not been used for small rural structures because of the high cost. These shallow profiles normally require steel and concrete together with a careful input of precision construction. They have to be built of materials which can take some bending. Masonry would not generally be used for such flat arches because it simply cannot sustain large bending moments. Masonry flat arches can become unstable with small lateral movements resulting from settlement at the springings. At the beginning of this study it was predicted that timber arches might succeed where other material could not.

The timber arches resist bending moments and are extremely good in compression. Through slip, they can settle into a minimal stress condition under dead load. Because dead load is uniform, only some of its compression ability, of which there is generally an excess, is used. This leaves the majority of the bending capacity for the live load, which produces the maximum effect at extreme loading conditions, as has been demonstrated above.

Although the focus was on flat arches, the study tested a range of shapes to quantify the trends of stress and deflection in order to broaden the result base, with a view to developing predictive analysis for arches for vehicle bridges. These arches would provide support to an upper deck, therefore the arch would take load at its centre. For that reason much of the testing was carried out with loading over the middle third of the arches.

The results show good correlation with elastic analysis, when lateral settlements, suitably modified modulus of elasticity values for external condition, modified cross sections etc. are taken into account. However, there are effects which are not readily accounted for in a traditional analysis, e.g. lateral tensions, so this research has aimed to address these issues in the next chapter and will conclude with an attempt to generate a semi empirical model.

The test results show timber arches to have a very high strength to weight ratio. There are optimum structural strengths and geometry for different uses. The shallow arch, for use as a pedestrian bridge, has a limit of 1 to 100, structural depth to span. This is based on limiting dynamics, where the natural frequency approaches the possible applied live load frequency. The optimum stiffness for load capacity comes from a steeper arch shown to have an approximate ratio of 1 to 5 for rise to span.

Perhaps the most useful outcome from the tests is confirmation that only approximately 1/3 of the initial prestress of $1N/mm^2$ is adequate for lateral load transfer. This satisfies Eurocode 5 Pt2 [51], but because of the locking effect and the end bearing actions, arches are much safer than stress laminated flat spans, at low lateral tension.

CHAPTER 8

8 DERIVATION OF SEMI EMPIRICAL MODEL AND DESIGN ISSUES

Although the tests detailed in Chapters 5, 6 and 7 have shown that SLT arches can be analysed elastically, these calculations include assumptions e.g. that the structure acts as an isotropic plate. This is not true if the lateral tension is low because the stiffness is reduced and not uniform. In order that this, and other effects, can be accounted for in some generic equations, a set of semi empirical models is developed in this chapter.

By observation and tests it was found that the load capacity of a SLT arch is mainly dependent on the following variables:

- span
- rise
- depth of section
- lateral tension
- timber grade – (characteristic stiffness in terms of modulus of elasticity and density)
- slip of laminates
- moisture content of timber
- settlement of supports
- loading conditions / type

The test programme in this research was carried out in order to show and evaluate relationships between these variables and a common denominator. All of the variables affect the stiffness of the structure, so that has been chosen as the common denominator and indicator of the performance of the structure. It is proposed to develop a partial empirical model to define any arch in terms of stiffness where stiffness, for the purposes of this study, is defined as the sustained load per unit of deformation. A design could then

be carried out by calculating a stiffness for the chosen parameters and comparing this to the required stiffness for the bridge being designed.
It is assumed that there is no significant interaction between some contributing factors listed above. In this regard these factors are studied independently.

One timber grade is chosen to develop relationships which are then factored by the relevant parameter for other grades in terms of modulus of elasticity and density. The stiffness chosen is load per unit of deflection so load becomes an external factor. Moisture content (MC) is eliminated by considering that all timbers are below 18% MC and because slip occurs with low lateral tension, it is regarded as part of that variable. The lateral tension effects are calculated from the test results provided by the sets of 2.1m and 6m span laboratory test arches and incorporated into the model. The generic semi empirical model as developed by Porteous and Kermani [83] is used and will take the form:-

$$K = F_1(G).F_2(T).F_3(E,D).F_4(\delta) \qquad \text{Equation 8.1}$$

Where:

K = Stiffness of the arch	load/unit deflection
F_1 = Geometric function(G)	made up of span (s), rise (r) and ring thickness (d)
F_2 = Lateral tension function (T)	force in the stressing bars at (b) centres
F_3 = Materials properties function (E, D)	modulus and density
F_4 = Horizontal settlement of supports (δ)	lateral movement

The stiffness, K, is taken as kN/mm which was the unit of experimental measurement.

8.1 Geometric Function $F_1(G)$

The geometric function includes three variables – span and rise and arch thickness. They are interdependent and will be chosen in any combination for a design. To develop a suitable relationship from test results alone would have required too many test structures so analytical results have been used. This is validated on the assumption that all tests showed good correlation with linear elastic analysis and where experimental results are available they will be compared in the conclusion to this chapter.

A series of parametric analyses, using QSE structural analysis software, was carried out to examine the effects on load deformation response with varying spans, rises and arch thicknesses. In the analyses the following were considered:-

1 Loads were applied as line loads across a 1m width of deck to simulate three different loading conditions:-
   i. Central point loading to simulate an arch supporting a structure above.
   ii. Quarter point load to simulate the most critical loading on an arch, This is the load applied if the arch ring is used as a bridge deck.
   iii. Two line loads at third span to simulate, approximately, uniformly distributed load and compare with test results.

2 Three different spans were considered. 6m, 10m, 20m.
3 Three or four different arch ring thicknesses were considered for each arch.
4 Four different rises were considered for each arch.
5 Lateral movement, settlement of supports, was not permitted.
6 Young's Modulus E was taken as $8.8kN/mm^2$ and weight density as $5kN/m^3$.

20kN of load was chosen to give realistic results for plotting but this is arbitrary because load is unitised in the resulting derivations. The effective width was taken as 0.75 times the actual width to account for the butt joints in the arch. SLT arches are generally made from laminates each containing four holes which in turn lead to four staggers in the joints. This means that at any cross section there is a butt joint at every fourth laminate therefore the effective cross section is 0.75 of the full cross section. The second moment of area in the vertical direction is also affected.

The following Table, 8.1, shows parametric stiffnesses calculated from deflections for the three arches of varying geometry for three different forms of 20kN of loading – single point load at mid span, single point load at quarter span and two 10kN point loads at third points. The gradient of the load deflection plots was taken as the stiffness which are shown as 'K' in Table 8.1. 'K' is the load which gives 1mm of deflection for each load and combination of span, rise and depth.

Table 8.1 – Parametric stiffnesses for three loadings

PARAMETRIC STIFFNESSES					Central Point Load K Parametric kN/mm	QTR Point Loading K Parametric kN/mm	Four Point Loading K Parametric kN/mm
Span mm	Depth mm	Rise mm	s/r	s/d			
20000	200	500	40.00	100.00	0.561	0.332	1.111
20000	200	1000	20.00	100.00	0.914	0.375	3.18
20000	200	2000	10.00	100.00	1.079	0.382	5.72
20000	200	3000	6.67	100.00	1.097	0.374	6.47
20000	200	4000	5.00	100.00	1.085	0.363	6.4
20000	200	5000	4.00	100.00	1.060	0.350	7.69
20000	200	10000	2.00	100.00	0.908	0.276	5.38
20000	250	500	40.00	80.00	0.855	0.600	1.46
20000	250	1000	20.00	80.00	1.591	0.714	4.66
20000	250	2000	10.00	80.00	2.030	0.740	9.59
20000	250	3000	6.67	80.00	2.103	0.729	11.5
20000	250	4000	5.00	80.00	2.094	0.708	11.83
20000	250	5000	4.00	80.00	2.053	0.682	14.08
20000	250	10000	2.00	80.00	1.770	0.539	10.26
20000	300	500	40.00	66.67	1.170	0.948	1.82
20000	300	1000	20.00	66.67	2.429	1.197	6.04
20000	300	2000	10.00	66.67	3.356	1.268	14.13
20000	300	3000	6.67	66.67	3.554	1.254	18.05
20000	300	4000	5.00	66.67	3.568	1.220	19.18
20000	300	5000	4.00	66.67	3.510	1.175	23
20000	300	10000	2.00	66.67	3.030	0.930	12.5
10000	100	250	40.00	100.00	0.560	0.352	1.11
10000	100	500	20.00	100.00	0.905	0.401	3.12
10000	100	1000	10.00	100.00	1.063	0.401	5.52
10000	100	1500	6.67	100.00	1.081	0.401	6.19
10000	100	2000	5.00	100.00	1.068	0.389	6.15
10000	100	3000	3.33	100.00	1.015	0.347	2.41
10000	100	4000	2.50	100.00	0.952	0.314	2.52
10000	150	250	40.00	66.67	1.174	0.973	3.45

10000	150	500	20.00	66.67	2.414	1.278	5.97
10000	150	1000	10.00	66.67	3.312	1.358	13.7
10000	150	1500	6.67	66.67	3.503	1.344	17.45
10000	150	2000	5.00	66.67	3.514	1.306	18.48
10000	150	3000	3.33	66.67	3.378	1.180	8.33
10000	150	4000	2.50	66.67	3.185	1.060	7.72
10000	200	250	40.00	50.00	1.862	1.810	2.6
10000	200	500	20.00	50.00	4.442	2.814	8.73
10000	200	1000	10.00	50.00	7.090	3.150	23.75
10000	200	1500	6.67	50.00	7.871	3.151	33.67
10000	200	2000	5.00	50.00	8.048	3.075	37.95
10000	200	3000	3.33	50.00	7.840	2.760	44.44
10000	200	4000	2.50	50.00	7.430	2.500	40.82
6000	75	100	60.00	80.00	0.500	0.480	0.713
6000	75	250	24.00	80.00	1.394	0.697	3.454
6000	75	500	12.00	80.00	1.914	0.740	8.032
6000	75	750	8.00	80.00	2.049	0.737	10.73
6000	75	1000	6.00	80.00	2.055	0.724	11.56
6000	75	1500	4.00	80.00	2.010	0.780	11.76
6000	75	2000	3.00	80.00	1.923	0.635	11.63
6000	100	100	60.00	60.00	0.779	0.900	1.026
6000	100	250	24.00	60.00	2.571	1.540	5.076
6000	100	500	12.00	60.00	4.107	1.719	13.89
6000	100	750	8.00	60.00	4.613	1.731	20.6
6000	100	1000	6.00	60.00	4.718	1.706	23.81
6000	100	1500	4.00	60.00	4.695	1.613	27.4
6000	100	2000	3.00	60.00	4.500	1.502	25.97
6000	150	100	60.00	40.00	1.470	1.946	1.823
6000	150	250	24.00	40.00	5.400	4.385	8.4
6000	150	500	12.00	40.00	10.935	5.488	26.67
6000	150	750	8.00	40.00	13.615	5.683	45.15
6000	150	1000	6.00	40.00	14.609	5.661	59.97
6000	150	1500	4.00	40.00	15.040	5.390	74.07
6000	150	2000	3.00	40.00	14.710	5.040	74.07
6000	200	100	60.00	30.00	2.43	3.4	3.03
6000	200	250	24.00	30.00	8.573	8.636	11.94
6000	200	500	12.00	30.00	20.101	12.107	39.06
6000	200	750	8.00	30.00	27.473	12.979	71.94
6000	200	1000	6.00	30.00	31.056	13.115	99
6000	200	1500	4.00	30.00	33.3	12.58	143
6000	200	2000	3.00	30.00	33.3	11.83	143

(the figures in red will be used later in an experimental correlation)

Stiffness was then plotted against the span/rise ratio for each individual combination of span and depth (thickness of arch ring). The plots for each loading case are shown below in Figures 8.1, 8.2 and 8.3:-

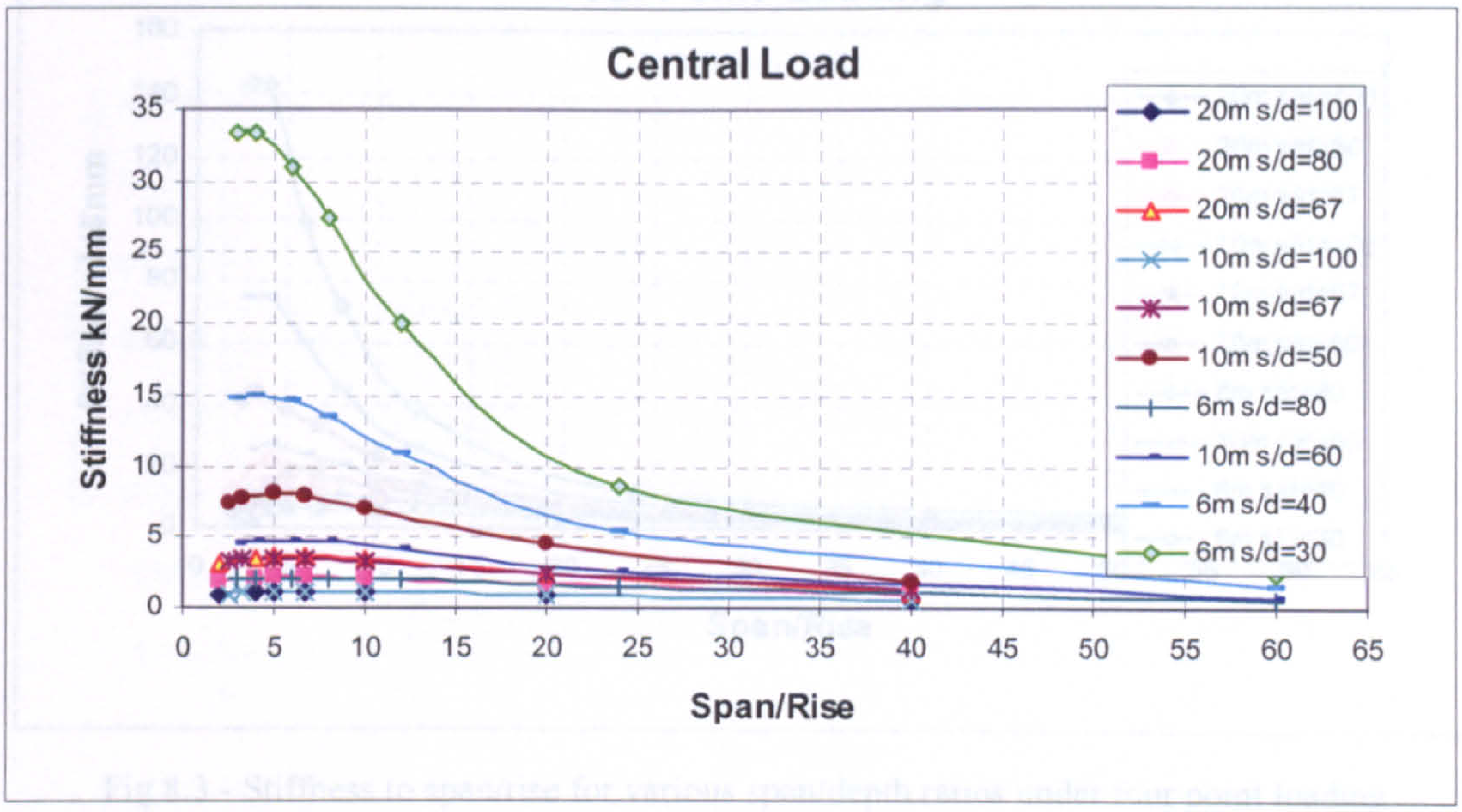


Fig 8.1 - Stiffness to span/rise for various span/depth ratios under central point load

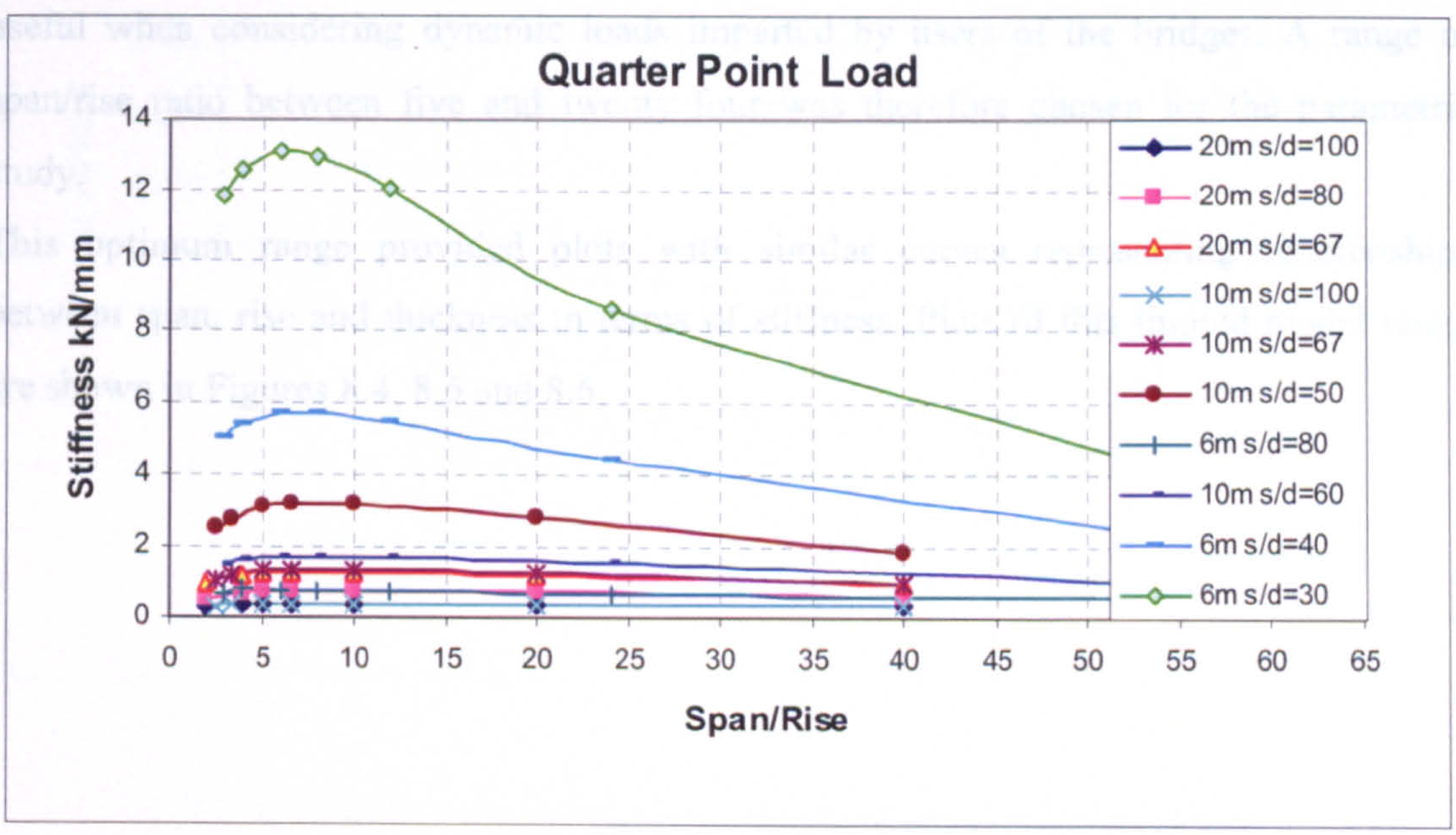


Fig 8.2 - Stiffness to span/rise for various span/depth ratios under quarter point load

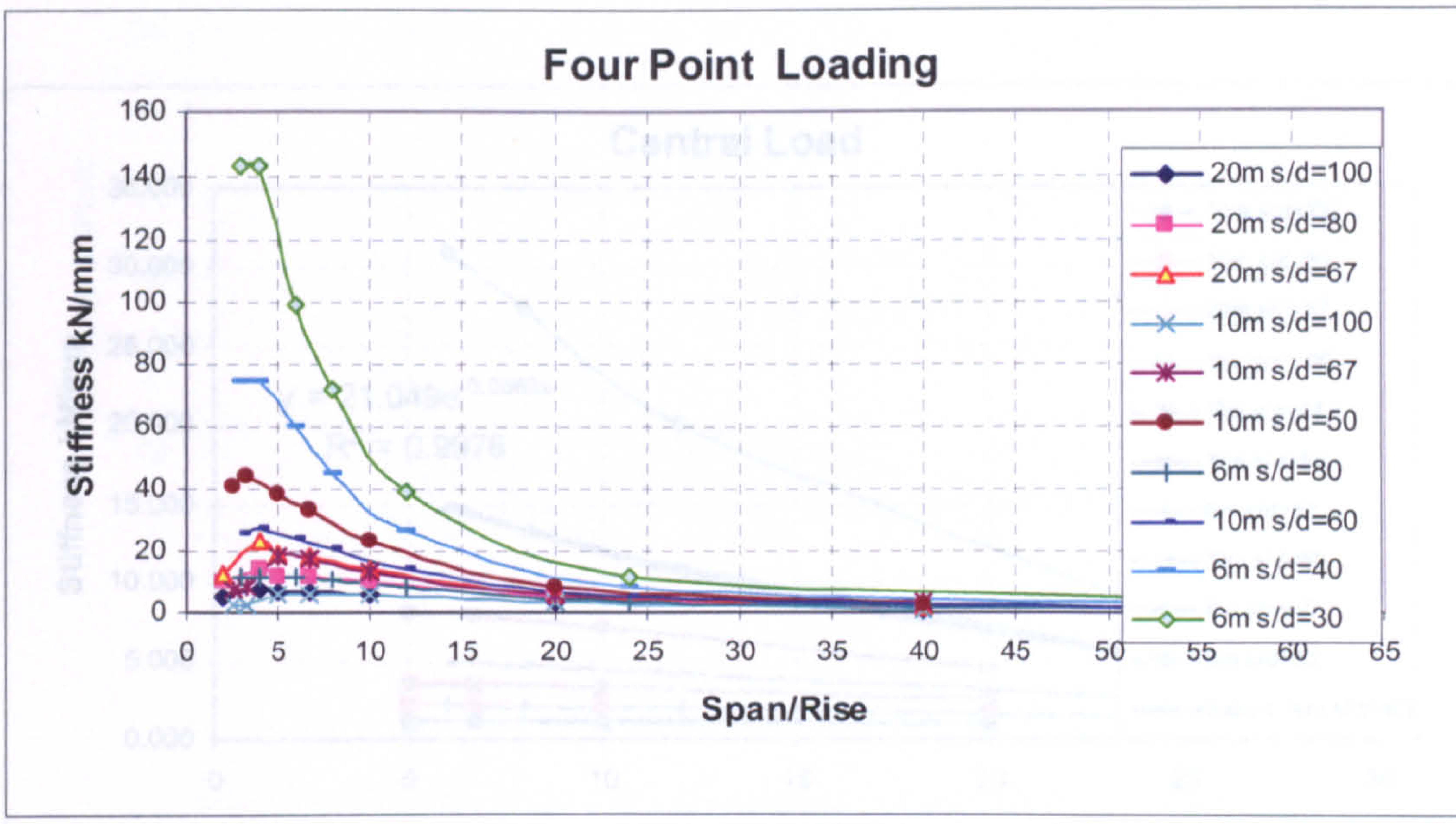


Fig 8.3 - Stiffness to span/rise for various span/depth ratios under four point loading

From the plots in Figures 8.1, 8.2 and 8.3 it can be seen that there is a common optimum span/rise ratio of approximately five. From test results, arches with span/rise ratios of greater than 24 had, or would have had, Fundamental Natural Frequencies too low to be useful when considering dynamic loads imparted by users of the bridges. A range of span/rise ratio between five and twenty four was therefore chosen for the parametric study.

This optimum range provided plots with similar curves representing relationships between span, rise and thickness in terms of stiffness. Plots of this limited model range are shown in Figures 8.4, 8.5 and 8.6.

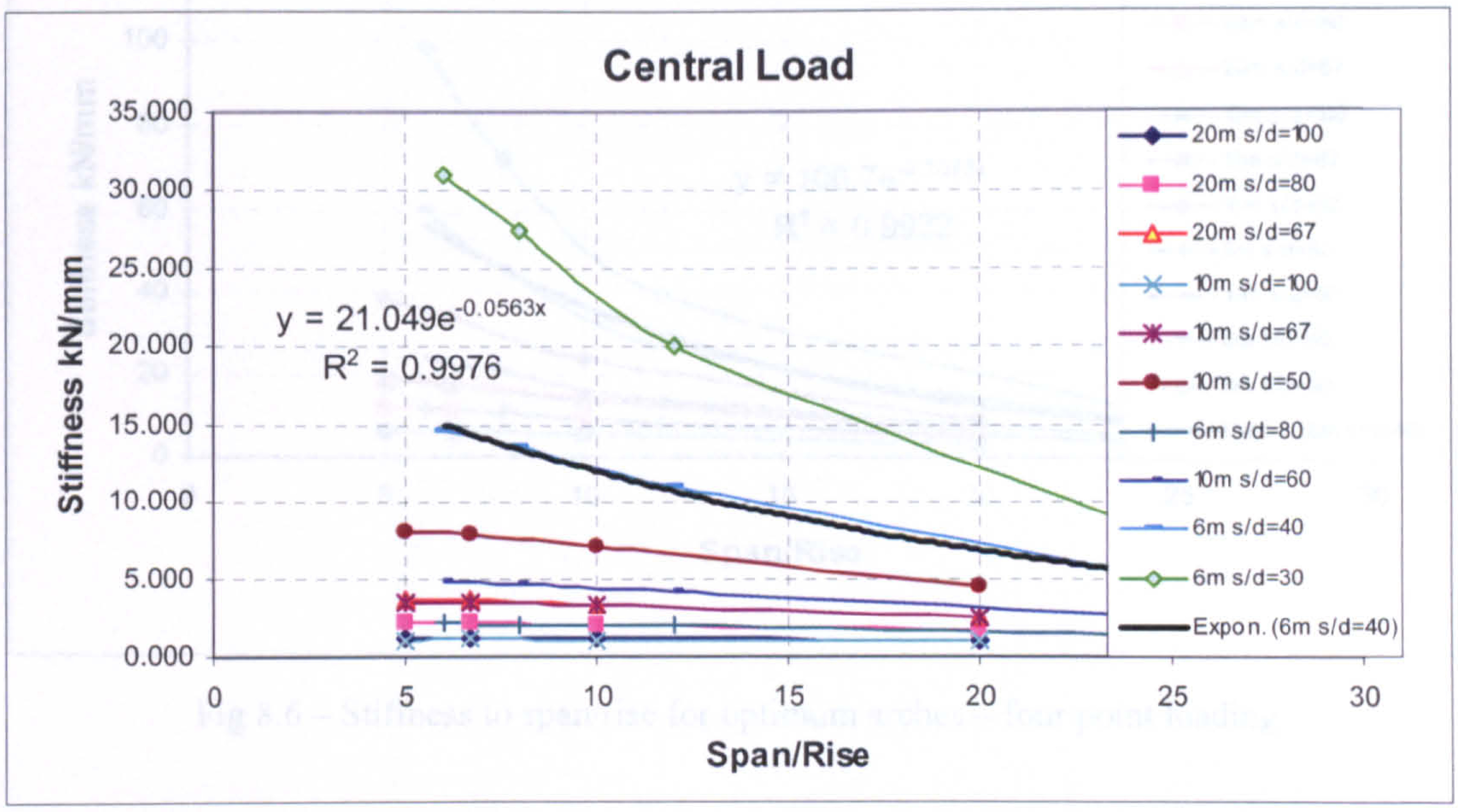


Fig 8.4 - Stiffness to span/rise for optimum arches – central point load

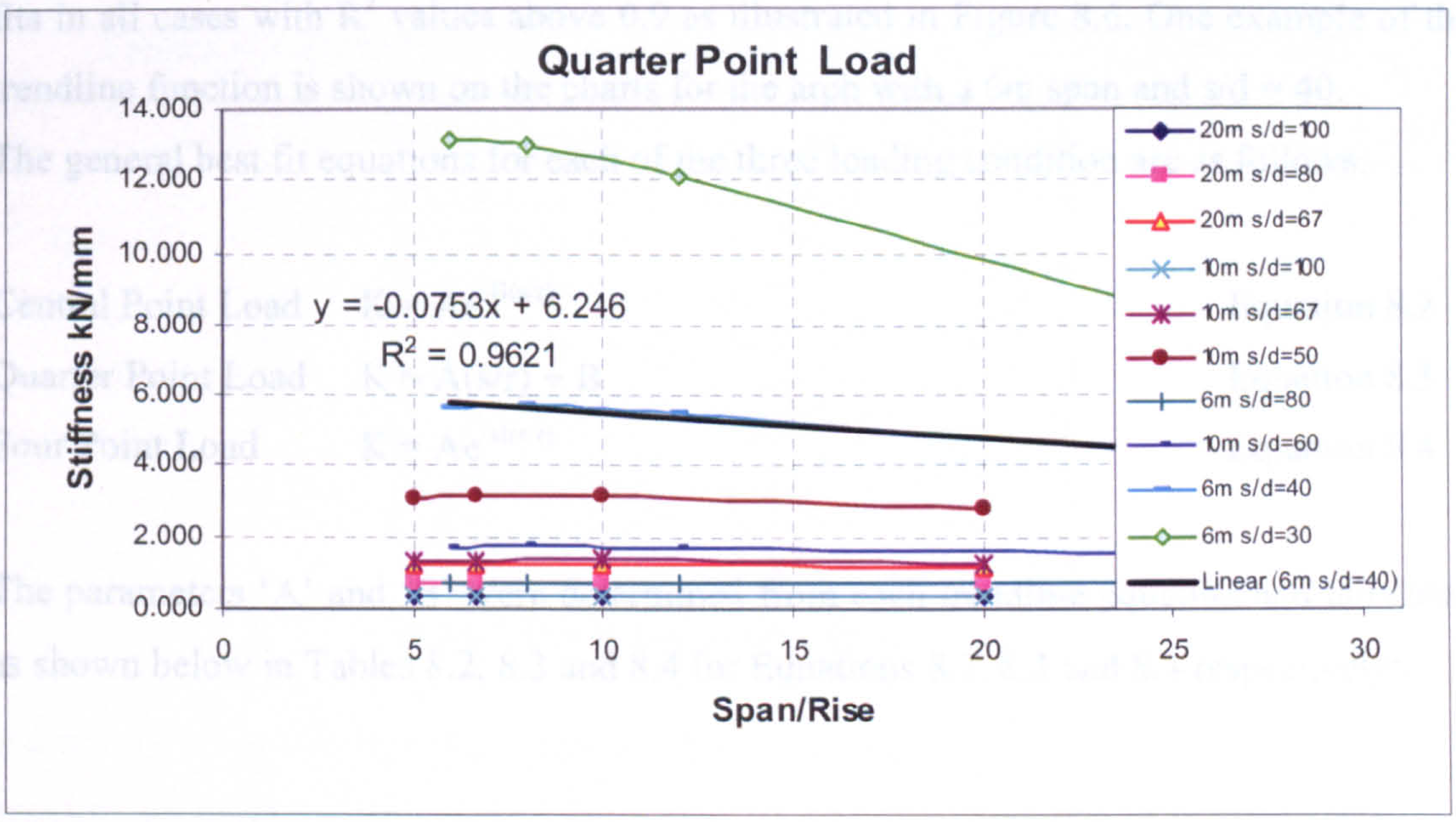


Fig 8.5 - Stiffness to span/rise for optimum arches – quarter point load

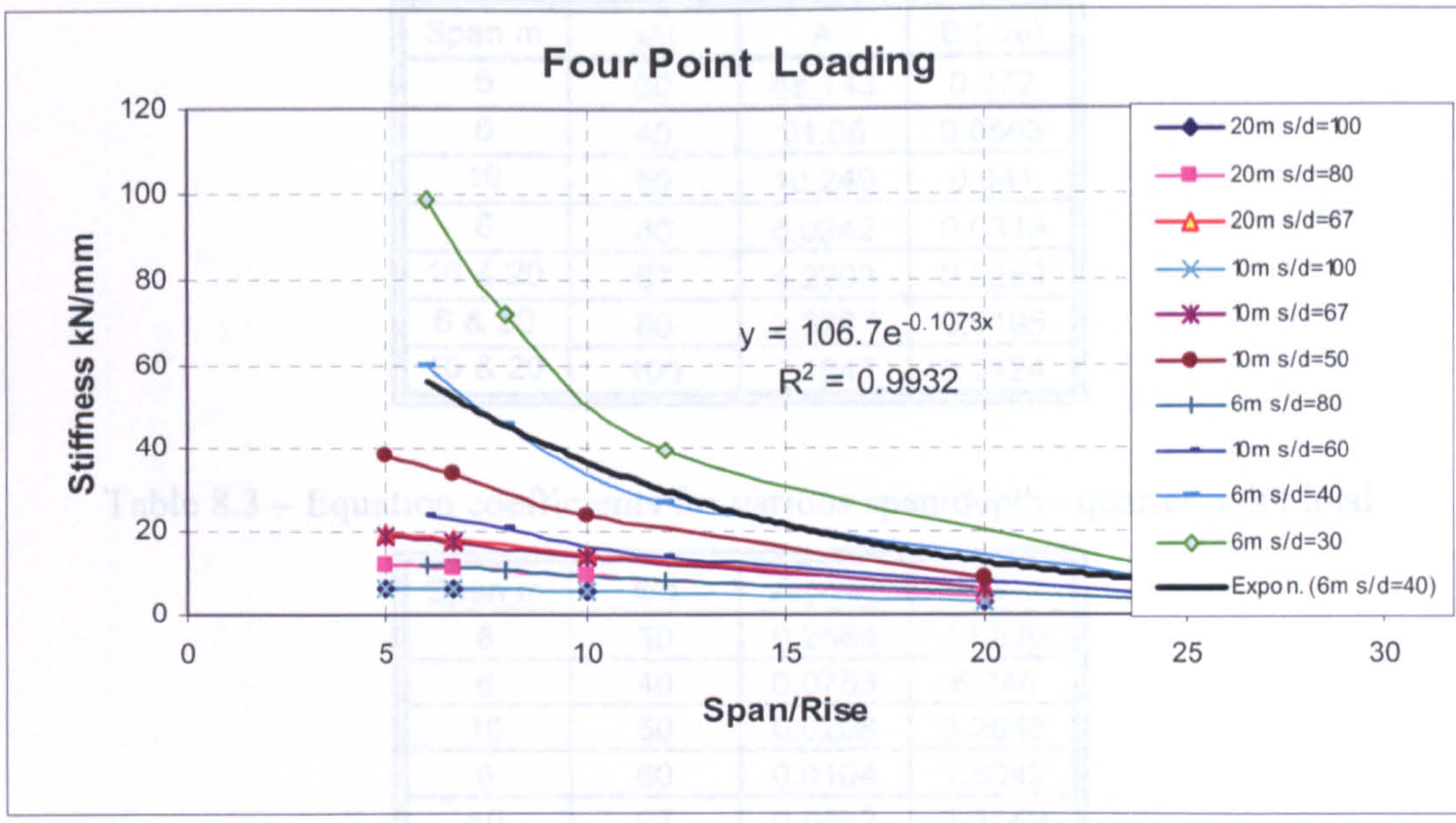


Fig 8.6 – Stiffness to span/rise for optimum arches – four point loading

The function which provided the best fit to the data was different for each loading case. Trendline functions provided in the spreadsheet, Excel, were used which provided good fits in all cases with R^2 values above 0.9 as illustrated in Figure 8.6. One example of the trendline function is shown on the charts for the arch with a 6m span and s/d = 40.

The general best fit equations for each of the three loading condition are as follows:-

Central Point Load	$K = Ae^{-B(s/r)}$	Equation 8.2
Quarter Point Load	$K = A(s/r) + B$	Equation 8.3
Four Point Load	$K = Ae^{-B(s/r)}$	Equation 8.4

The parameters 'A' and 'B' were determined from each trendline equation and tabulated as shown below in Tables 8.2, 8.3 and 8.4 for Equations 8.2, 8.3 and 8.4 respectively:-

Table 8.2 – Equation coefficients for various span/depth – central point load

Span m	s/d	A	B (-ve)
6	30	48.143	0.072
6	40	21.05	0.0563
10	50	10.249	0.041
6	60	6.0242	0.0349
10 & 20	67	4.2208	0.0269
6 & 20	80	2.3787	0.0195
10 & 20	100	1.1847	0.0124

Table 8.3 – Equation coefficients for various span/depth– quarter point load

Span m	s/d	A (-ve)	B
6	30	0.2584	14.939
6	40	0.0753	6.246
10	50	0.0208	3.2643
6	60	0.0104	1.8042
10	67	0.0032	1.3549
20	67	0.0027	1.2632
6	80	0.0003	0.7254
20	80	0.002	0.7494
10	100	0.0005	0.393
20	100	0.0004	0.3688

Table 8.4 – Equation coefficients for various span/depth – four point loading

Span m	s/d	A	B (-ve)
6	30	179.72	0.115
6	40	106.7	0.1073
10	50	63.787	0.0992
6	60	40.169	0.0864
10	67	29.901	0.0791
20	67	28.556	0.0775
6	80	17.289	0.0645
20	80	18.052	0.0686
10	100	8.7689	0.0495
20	100	8.3317	0.0479

The values of the coefficients, A and B were plotted against their span/depth ratios to provide two plots for each loading condition. These plots are shown in Figures 8.7, 8.8 and 8.9.

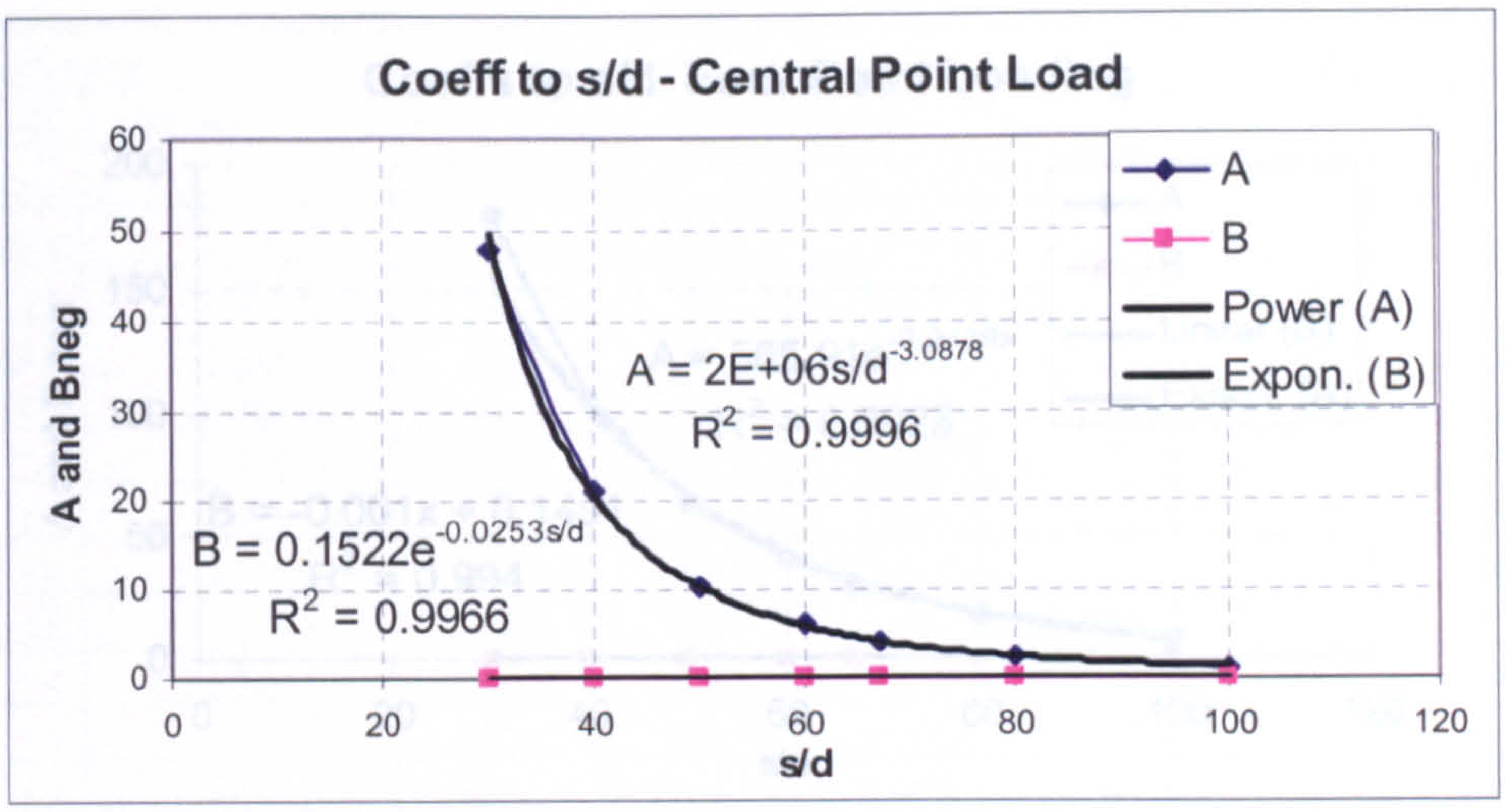


Fig 8.7 – Coefficients to span/depth –central point load

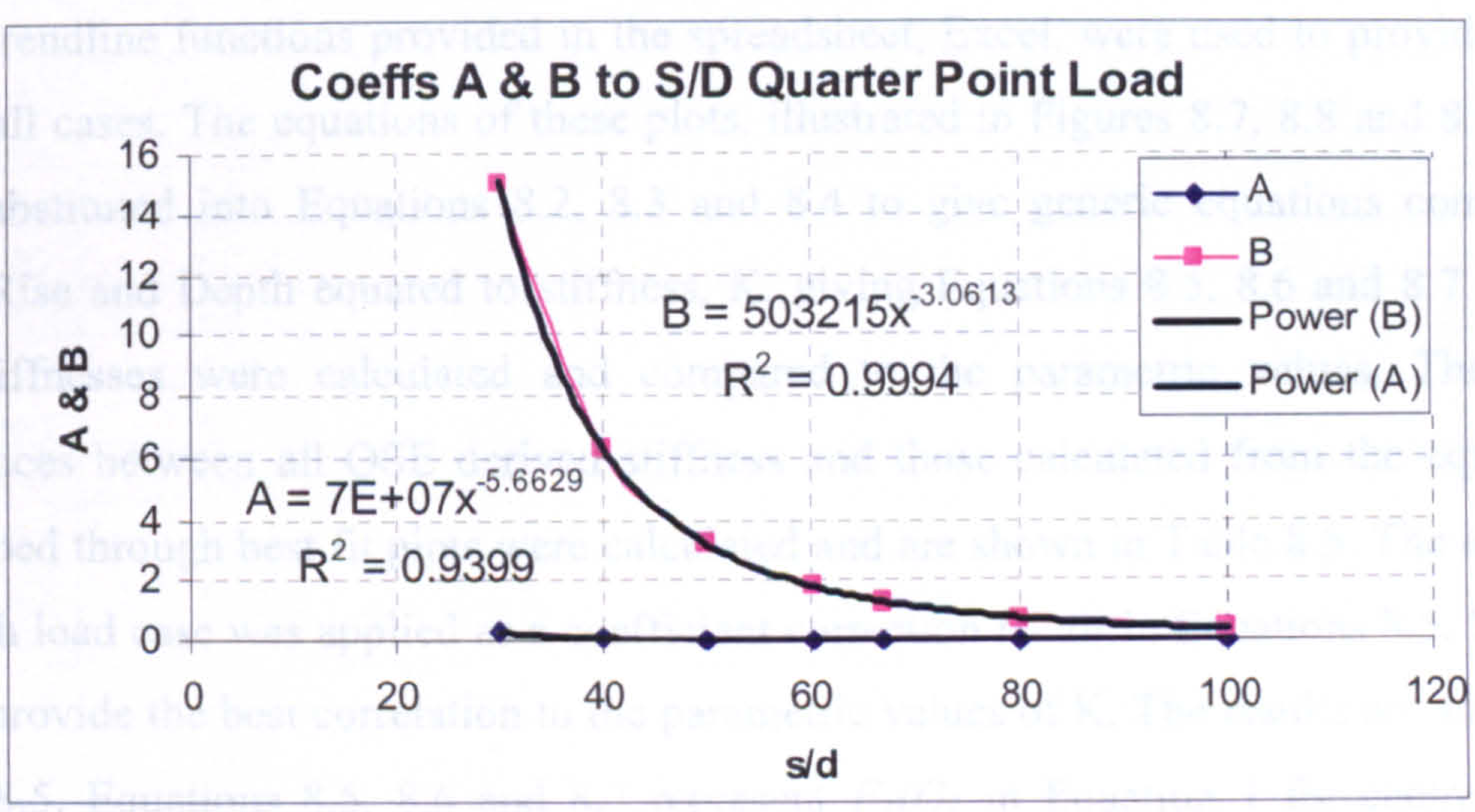


Fig 8.8 – Coefficients to span/depth – quarter point load

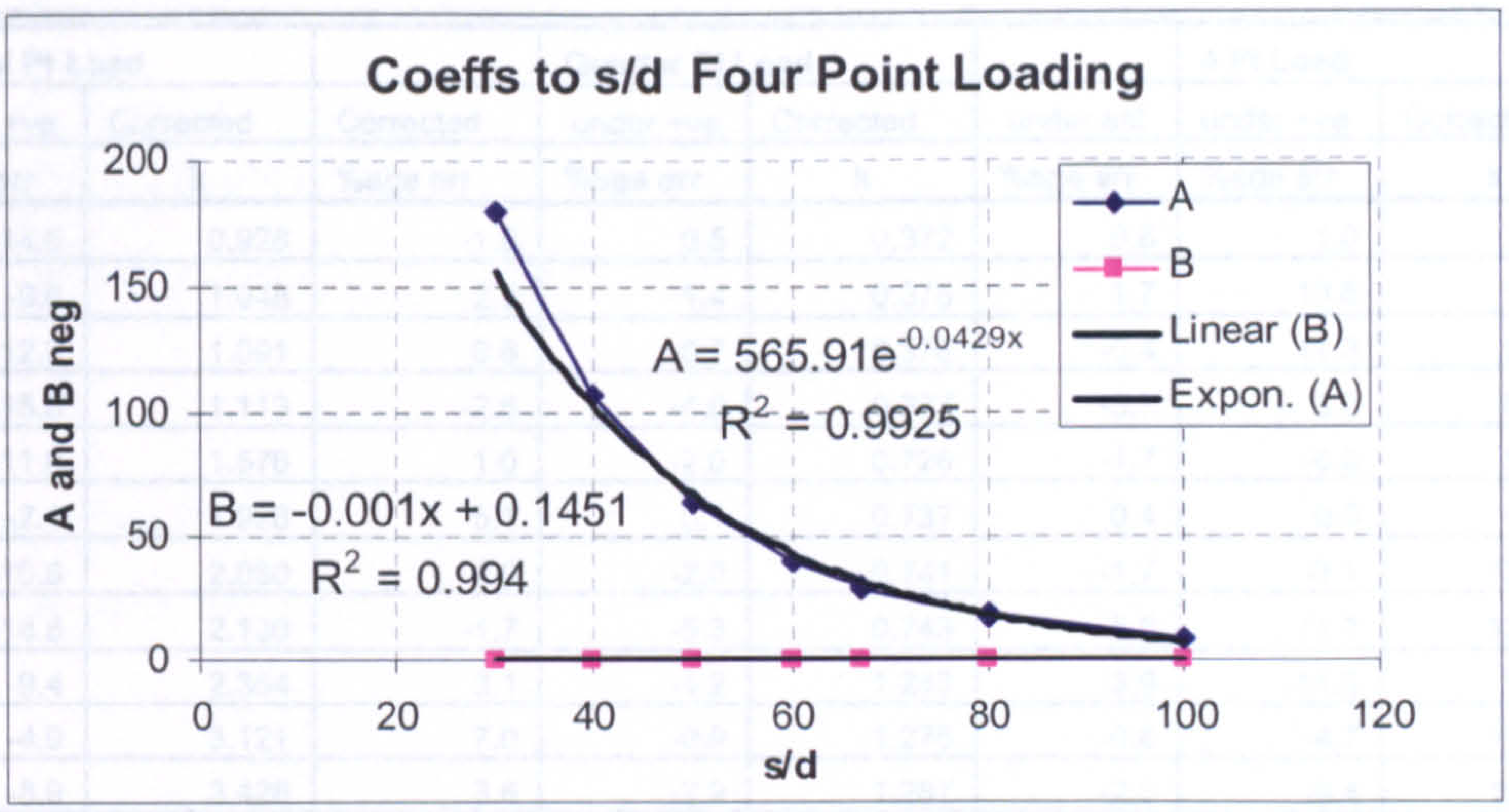


Fig 8.9 – Coefficients to span/depth –four point loading

The function which provided the best fit to the data was again different for each loading case. Trendline functions provided in the spreadsheet, Excel, were used to provide good fits in all cases. The equations of these plots, illustrated in Figures 8.7, 8.8 and 8.9 were then substituted into Equations 8.2, 8.3 and 8.4 to give generic equations containing Span, Rise and Depth equated to stiffness, K, giving Equations 8.5, 8.6 and 8.7 below. The stiffnesses were calculated and compared to the parametric values. The error differences between all QSE derived stiffness and those calculated from the equations developed through best fit plots were calculated and are shown in Table 8.5. The average for each load case was applied as a coefficient correction factor in Equations 8.5, 8.6 and 8.7 to provide the best correlation to the parametric values of K. The results are shown in Table 8.5. Equations 8.5, 8.6 and 8.7 represent $F_1(G)$ in Equation 1 for central point loading, quarter point loading and four point loading respectively.

Table 8.5 – Error difference - Generic equation stiffnesses and parametric values for 's/r' range 5 – 24

Arch	**Central Pt Load**			**Quarter Pt Load**			**4 Pt Load**		
Span	under +ve	Corrected	Corrected	under +ve	Corrected	under est	under +ve	Corrected	Corrected
depth/rise	%age err	**k**	%age err	**%age err**	**k**	**%age err**	%age err	**k**	%age err
20/0.2/1	-14.6	0.928	-1.5	0.5	0.372	0.8	1.0	3.226	-1.4
20/0.2/2	-9.6	1.048	2.9	1.4	0.375	1.7	13.6	5.064	11.5
20/0.2/3	-12.2	1.091	0.6	-0.7	0.376	-0.4	11.3	5.885	9.0
20/0.2/4	-15.8	1.113	-2.6	-4.0	0.377	-3.7	3.3	6.345	0.9
20/0.25/1	-11.8	1.576	1.0	-2.0	0.726	-1.7	-6.8	5.099	-9.4
20/0.25/2	-7.1	1.926	5.1	0.1	0.737	0.4	0.5	9.778	-2.0
20/0.25/3	-10.6	2.060	2.0	-2.0	0.741	-1.7	-3.1	12.148	-5.6
20/0.25/4	-14.8	2.130	-1.7	-5.3	0.743	-5.0	-11.7	13.540	-14.5
20/0.3/1	-9.4	2.354	3.1	-4.2	1.243	-3.9	-11.8	6.920	-14.6
20/0.3/2	-4.9	3.121	7.0	-0.9	1.276	-0.6	-4.7	15.162	-7.3
20/0.3/3	-8.9	3.428	3.6	-2.9	1.287	-2.6	-6.4	19.693	-9.1
20/0.3/4	-13.6	3.593	-0.7	-6.3	1.293	-6.0	-14.2	22.443	-17.0
10/0.1/0.5	-15.7	0.928	-2.5	7.0	0.372	7.3	-0.9	3.226	-3.4
10/0.1/1	-11.2	1.048	1.4	6.1	0.375	6.4	10.5	5.064	8.3
10/0.1/1.5	-13.9	1.091	-0.9	6.0	0.376	6.3	7.2	5.885	4.9
10/0.1/2	-17.6	1.113	-4.2	2.9	0.377	3.1	-0.7	6.345	-3.2
10/.15/0.5	-10.1	2.354	2.5	2.4	1.243	2.7	-13.1	6.920	-15.9
10/.15/1	-6.3	3.121	5.8	5.8	1.276	6.0	-8.0	15.162	-10.7
10/.15/1.5	-10.4	3.428	2.1	3.9	1.287	4.2	-10.1	19.693	-12.9
10/.15/2	-15.4	3.593	-2.2	0.7	1.293	1.0	-18.5	22.443	-21.4
10/0.2/0.5	-8.2	4.259	4.1	-0.7	2.824	-0.4	-13.3	10.136	-16.1
10/0.2/1	-4.2	6.544	7.7	4.8	2.991	5.0	-7.8	26.236	-10.5
10/0.2/1.5	-8.3	7.551	4.1	3.0	3.047	3.3	-4.4	36.022	-7.0
10/0.2/2	-13.8	8.112	-0.8	-0.3	3.074	0.0	-8.5	42.209	-11.2
6/.075/.25	-17.7	1.454	-4.3	-3.8	0.721	-3.5	-11.0	3.930	-13.8
6/.075/.5	-9.1	1.851	3.3	0.4	0.735	0.7	-4.3	8.584	-6.9
6/.075/.75	-10.5	2.006	2.1	-0.6	0.740	-0.3	-1.3	11.138	-3.8
6/.075/1	-14.7	2.088	-1.6	-2.8	0.742	-2.5	-7.1	12.686	-9.7
6/0.1/.25	-12.9	2.572	0.0	-8.4	1.664	-8.1	-10.2	5.736	-13.0
6/0.1/.5	-5.5	3.838	6.6	-1.3	1.736	-1.0	-11.9	15.926	-14.7
6/0.1/.75	-7.3	4.385	4.9	-2.0	1.760	-1.7	-6.0	22.384	-8.7
6/0.1/1	-12.1	4.688	0.6	-4.1	1.771	-3.8	-8.7	26.537	-11.5
6/0.15/.25	-11.0	5.309	1.7	-10.6	4.835	-10.3	2.8	8.371	0.3
6/0.15/.5	-6.4	10.311	5.7	-1.3	5.544	-1.0	-8.1	29.545	-10.8
6/0.15/.75	-6.6	12.865	5.5	-2.0	5.780	-1.7	2.8	44.985	0.4
6/0.15/1	-11.0	14.370	1.6	-4.5	5.898	-4.2	9.7	55.508	7.4
6/0.2/.25	-15.9	8.806	-2.7	8.8	7.853	9.1	17.4	10.112	15.3
6/0.2/.5	-16.3	20.706	-3.0	5.0	11.469	5.3	-0.5	40.243	-3.0
6/0.2/.75	-13.1	27.533	-0.2	2.1	12.674	2.3	13.5	63.773	11.4
6/0.2/1	-15.4	31.750	-2.2	-1.5	13.277	-1.2	20.9	80.281	18.9
	Ave - 11.4 ie .886		Ave =1.3	**Ave = -0.3 ie .997**		Ave = 0	**Ave = -2.5 ie .975**		Ave =0.1

Central Point Loading

$$K = 0.886\,(2E + 6(s/d)^{-3.0878})\,e^{-(0.1522\,e^{-0.0253\,(s/d)})(s/r)}$$

Equation 8.5

Quarter Point Loading

$$K = 0.997(7E + 7(s/d)^{-5.6629}(s/r) + 503215(s/d)^{-3.0613})$$

Equation 8.6

Four Point Loading

$$K = 0.975\,(565.91\,e^{-0.0429\,(s/d)})\,e^{-(-0.001\,(s/d)+0.1451\,)(s/r)}$$

Equation 8.7

It can be seen that the correlations for the Quarter Point load and the Four Point load are very close – average errors 0.3% and 2.5%. The central point loading correlation contains acceptable errors but they are of the order of twice the percentage of the others. This four point loading mirrors the loading used in the laboratory tests on the series of 2.1m spans and 6m spans, so the equation will be used to compare parametric results to test results in 8.6.

8.2 Lateral Tension Function $F_2(T)$

As detailed in Chapter 7, a series of tests was carried out to determine the effects of the level of tension (transverse stress) on the load deformation behaviour of 2.1m and 6.0m span arches Figure 8.10, with different rises (from flat to nearly semi circular). Linear regression was applied to each plot e.g. Figures 7.10 – 7.13, to determine the slope of the load / deformation characteristics for each tension level to represent the arch stiffness. These stiffness values were plotted against their lateral tension as shown in Figures 8.11 and 8.12. All of the plots show maximum stiffness is gained at relatively low tensions. Much higher tensions are recommended for flat stress laminated bridge decks in Eurocode 5 as well as codes in USA and Australia. However, as the lower limit for structural safety is given as 30% of design tension, test results will be used to derive a factor for the semi empirical model to modify stiffness for tensions below 30%. This is done by deriving a general equation fit for plots below minimum tension and applying it to the model.

Fig 8.10(a) – 6m span arch tested for varying lateral tensions

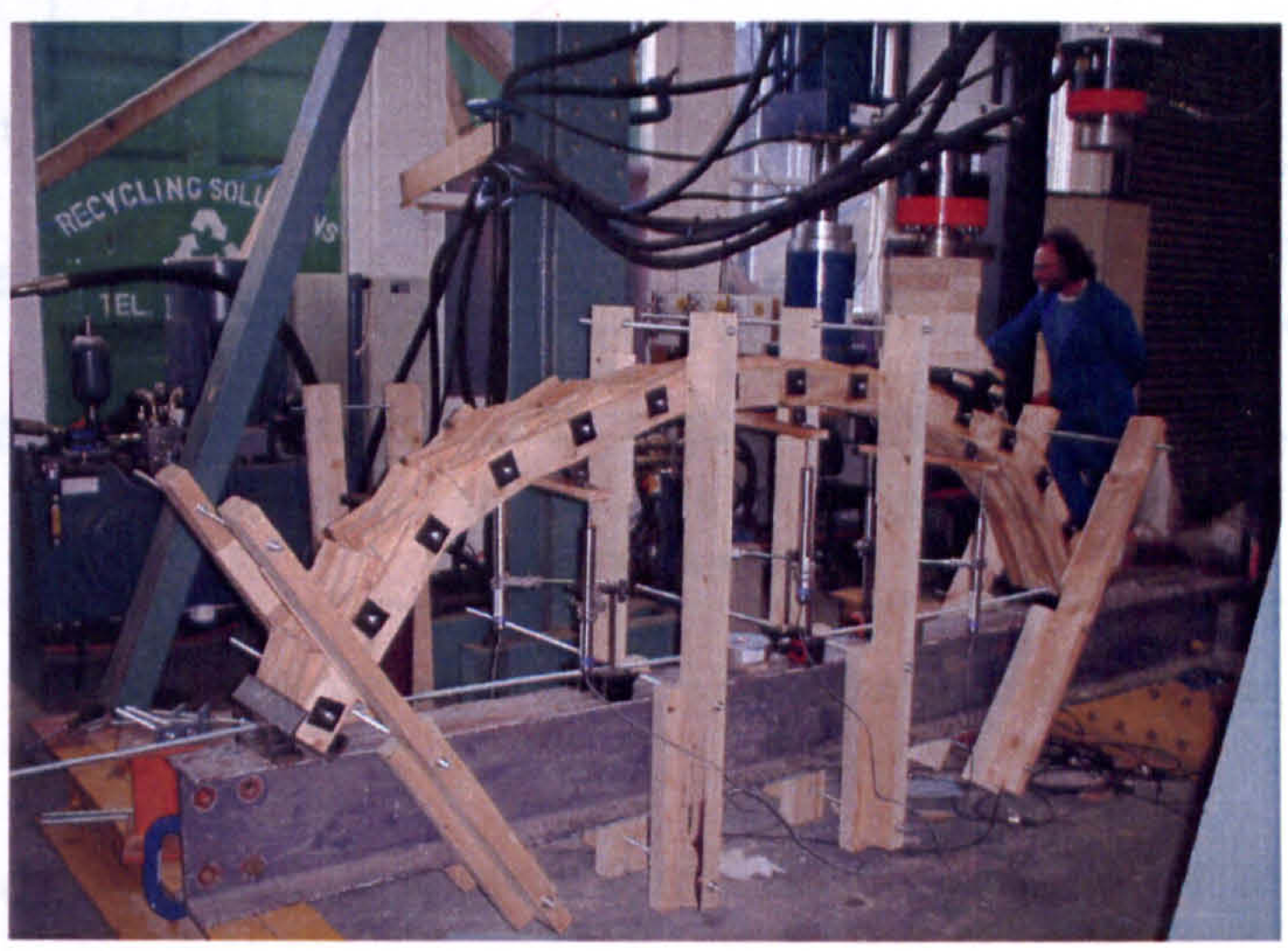


Fig 8.10(b) – 2.1m span arch tested for varying lateral tensions

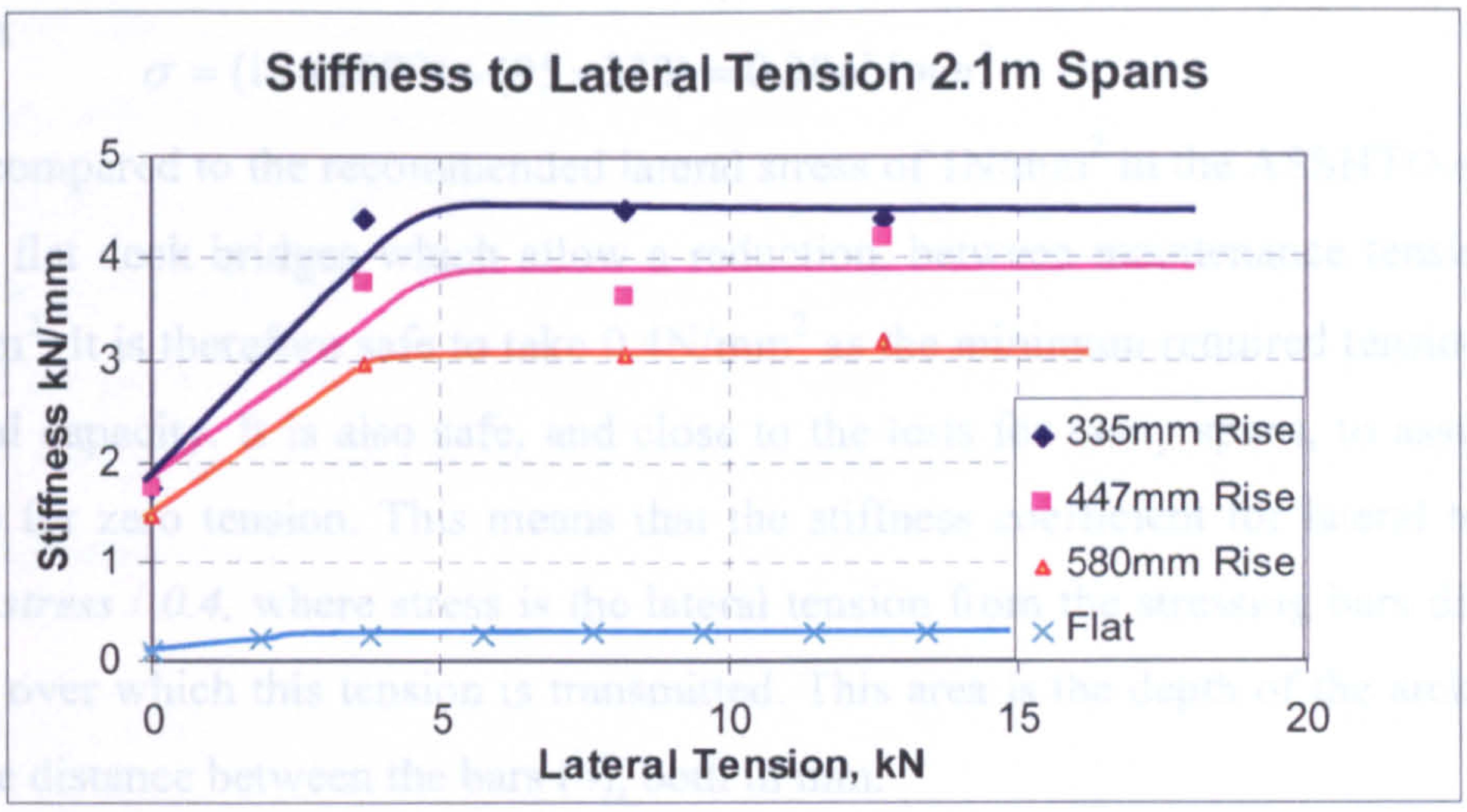


Fig 8.11 – Maximum stiffness to tension – 2.1m spans

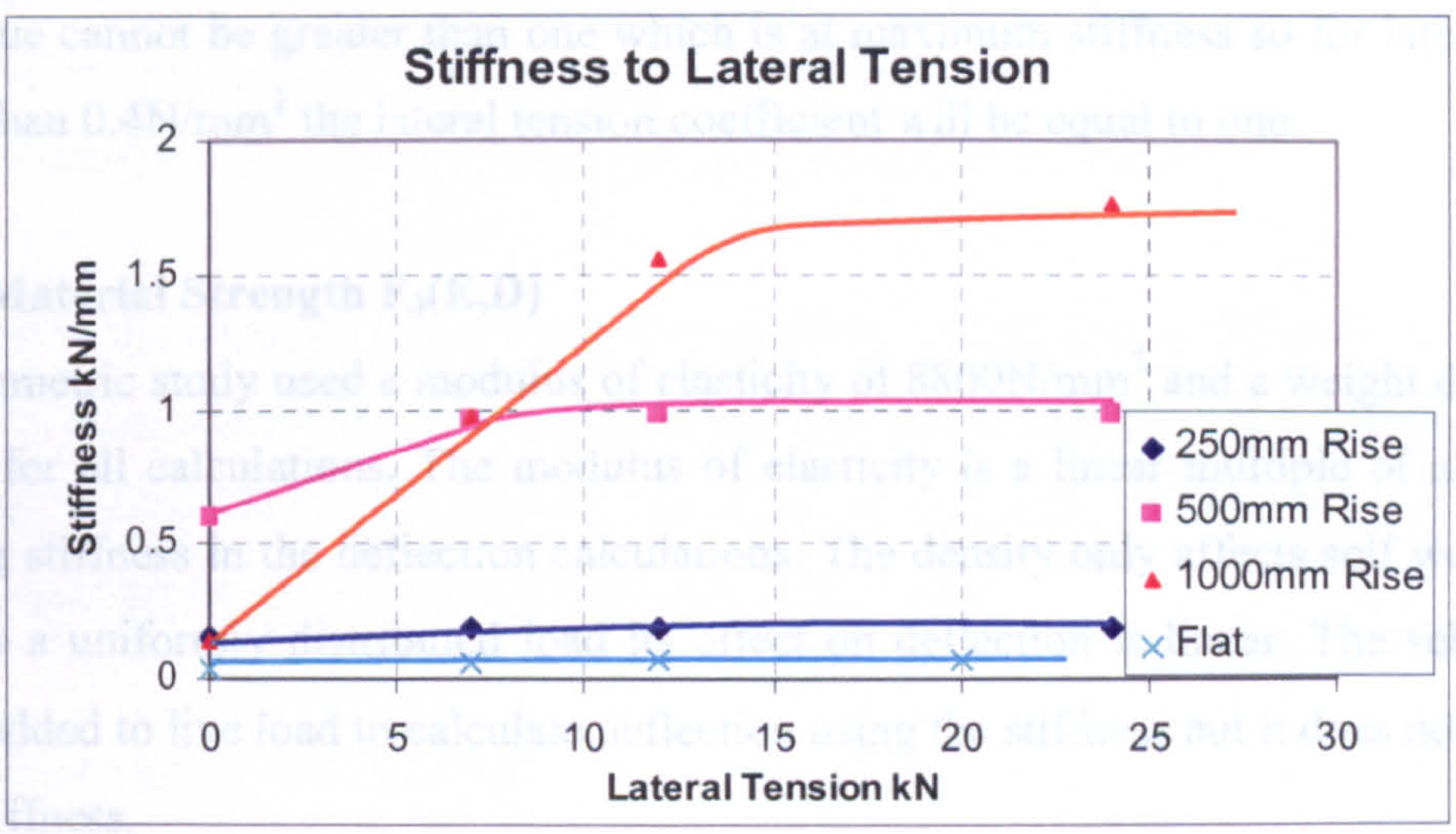


Fig 8.12 – Maximum stiffness to lateral tension – 6m spans

From Figures 8.11 and 8.12, it is clear that sufficient tension for a working arch is reached at 5kN for the 2.1m span arch and similarly 12kN for the 6m arch for all rises. These tensions give lateral stresses between the laminates of 0.4 and 0.38 N/mm^2 respectively.

2.1m Span $\sigma = (5 \times 1000) \div (70 \times 177) = 0.4 N/mm^2$

6m Span
$$\sigma = (12 \times 1000) \div (95 \times 333) = 0.38 N/mm^2$$

This is compared to the recommended lateral stress of 1N/mm^2 in the ASSHTO standards [41] for flat deck bridges which allow a reduction, between maintenance tensioning, to 0.3N/mm^2. It is therefore safe to take 0.4N/mm^2 as the minimum required tension for full structural capacity. It is also safe, and close to the tests for many spans, to assume zero stiffness for zero tension. This means that the stiffness coefficient for lateral tension is *applied stress / 0.4,* where stress is the lateral tension from the stressing bars divided by the area over which this tension is transmitted. This area is the depth of the arch ring (d) times the distance between the bars (b), both in mm.

$$F(T) = T/(b \times d)/0.4 \qquad \text{Equation 8.8}$$

This value cannot be greater than one which is at maximum stiffness so for lateral stress greater than 0.4N/mm^2 the lateral tension coefficient will be equal to one.

8.3 Material Strength $F_3(E,D)$

The parametric study used a modulus of elasticity of 8800N/mm^2 and a weight density of 5kN/m^3 for all calculations. The modulus of elasticity is a linear multiple of all factors affecting stiffness in the deflection calculations. The density only affects self weight and hence as a uniformly distributed load its effect on deflection is linear. The self weight will be added to live load to calculate deflection using the stiffness but it does not directly affect stiffness.

Therefore:

$$F_3(E,D) = E/8800 \qquad \text{Equation 8. 9}$$

where E is in N/mm^2

8.4 Settlement Function $F_4(\delta)$

Settlement of the arch springings creates a load effect which results in a vertical deflection and forces in the arch members. Whatever the arch is made of the vertical deflection will be geometrically proportional to the settlement. In an arch made of high modulus of elasticity material such as steel, the vertical deflection, for specific lateral settlement, will be the same as for a low modulus of elasticity material such as timber. The forces in the steel members will be greater as will the lateral thrust to create that same settlement. The horizontal settlement is entirely dependent on the lateral support provided for the arch and is independent of the characteristics of the arch. For that reason it will be treated as an independent additional deflection in the semi empirical model as shown in Equation 8.10.

$$P/(\Delta_{st} + \delta_{se}) = K \qquad \text{Equation 8.10}$$

Where

P = the applied load

Δ_{st} = the vertical deflection of the arch due to structural effects

δ_{se} = the vertical settlement due to the horizontal settlement

K = the stiffness of the arch in terms of load per unit of deflection

Settlement at the supports is generally very small in comparison to the span as is shown below by a parametric study. The increased stresses in the laminates are very small when settlement occurs. That is, the stresses calculated in, say, a 20m span compared to those in a 20m span plus a few millimetres added for settlement.

Vertical load causes vertical deflection which usually results in some lateral settlement in practice. Using geometry, a vertical deflection in an un-tensioned arch, can be calculated, because it relates directly to lateral movement. It is shown below in a parametric comparison that the increase in vertical deflection, under load, when lateral settlement is allowed, is almost all due to the lateral movement. Here, this is named 'geometric deflection' as opposed to 'structural deflection' which is caused by loads deforming the structure.

Vertical deflection is therefore made up of two parts as described in Equation 8.10:-

1 Structural deflection from strain in the structure

2 Geometric deflection resulting from lateral settlement

To describe this, a 20m span arch was evaluated using linear elastic analysis to calculate deflections and forces resulting from a pedestrian loading. The 20m span was chosen because a test bridge was built with that span and the horizontal settlement of approximately 8mm (Section 6.9.7) was observed under load.

Central deflection for full loading and zero settlement (QSE)	= 10.054mm
Central deflection for full loading with 8mm settlement (QSE)	= 29.011mm
Force parallel to grain at the supports, zero settlement (QSE)	= 123kN
Force parallel to grain at the supports with 8mm of settlement (QSE)	= 122kN

Therefore:

1 Because of the low modulus of elasticity of timber and the stress release capability of the stress-laminated construction, additional stress in the laminates from the force parallel to the grain is negligible.

2 Additional deflection due to geometry and some structural effects

= 29.011-10.054 = 18.957mm

3 Calculated geometric vertical deflection from an 8mm lateral settlement in a 1 to 12 secular arch = 19.36mm (Excel spreadsheet, Appendix 10)

4 Nearly all of the vertical deflection is due to the lateral settlement

5 The difference between the additional deflection in the loaded arch and the calculated geometric deflection

=19.36-18.957 =0.403mm

Is due to the structural resistance.

The additional structural effects from settlement are very small because of the low modulus of elasticity of timber but also due to some slip between laminates. In practice there will be a very small slip between laminates allowing the arch to take up a new shape. This new arch, with the induced stresses released, will then resist the load effects.

The comparative effects of lateral settlement are illustrated by the following parametric study of two sets of 6m span arches, with 250mm, 500mm, 1000mm and 1500mm rises and 100mm deep sections, one made from timber and the other from steel. Table 8.6 shows the results of deflection and maximum arch barrel forces, at the springing, for zero load and 1mm of lateral settlement.

Table 8.6 – Deflections and forces for arches with zero load and 1mm settlement

	Timber		Steel	
Arch Rise mm	Vertical Deflection mm	Maximum Member Force kN	Vertical Deflection mm	Maximum Member Force kN
250	4.6	2.71	4.6	63.09
500	2.34	0.686	2.34	15.98
1000	1.1.34	0.168	1.134	3.93
1500	0.778	0.072	0.778	1.687

It can be seen that vertical deflection is geometrically proportional to span and rise and is exactly the same for steel and timber for the same settlement. Forces increase proportionately with increasing modulus of elasticity. The size of the forces induced in the timber arch by settlement is relatively small and therefore can be ignored in the semi empirical model.

Therefore:

To account for lateral settlement in the semi empirical model when using it to design an arch:-

1. Assess the lateral settlement likely to occur in practice
2. Calculate the geometric vertical deflection the settlement will induce
3. Add that deflection to the deflection calculated from the stiffness equation
4. The resulting deflection is the total deflection of the arch. (Equation 8.10)

8.5 Generic Formula – Proposed Semi Empirical Equation

Combining equations (5), (6), (7), (8), and (9)

Equation 8.5 becomes

Central Point Loading

$$K = 0.886(2\times10^{6}(s/d)^{-3.0878})e^{-(0.1522e^{-0.0253(s/d)})(s/r)}(T/bd/0.4)(E/8800)$$

Equation 8.11

Quarter Point Loading

$$K = 0.997(7\times10^{7}(s/d)^{-5.6629}(s/r)+503215(s/d)^{-3.0613})(T/bd/0.4)(E/8800)$$

Equation 8.12

Four Point Loading

$$K = 0.975(565.91e^{-0.0429(s/d)})e^{-(-0.001(s/d)+0.1451)(s/r)}(T/bd/0.4)(E/8800)$$

Equation 8.13

Where

s = span, mm

d = depth of laminate, mm

r = rise of arch, mm

b = distance between tension bars, mm

T = tension in bars, N

E = modulus of elasticity, N/mm^2

If it is required to consider a structure with uniformly distributed loadings the loads can be converted to equivalent four point loading so that Equations 8.13 can be used.

Table 8.7 shows stiffnesses calculated from the proposed equations. The modulus of elasticity was not altered from that used to derive values in Table 8.1 ($E = 8800N/mm^2$). Full lateral tension has been assumed and no lateral settlement was allowed for.

Table 8.7 – Stiffnesses calculated from proposed Equations 8.12, 8.13 and 8.14

Arch		20m Span			10m Span			6m Span		
		Central Pt Load	Quarter Pt Load	4 Pt Load	Central Pt Load	Quarter Pt Load	4 Pt Load	Central Pt Load	Quarter Pt Load	4 Pt Load
Depth mm	Rise mm	K kN/mm	K kN/mm	K kN/mm	K kN/mm	K kN/mm	K kN/mm	K kN/mm	K kN/mm	K kN/mm
75	250							1.454	0.721	3.930
75	500							1.851	0.735	8.584
75	750							2.006	0.740	11.138
75	1000							2.088	0.742	12.686
100	250							2.572	1.664	5.736
100	500				.928	.372	3.226	3.838	1.736	15.926
100	750							4.385	1.760	22.384
100	1000				1.048	.375	5.064	4.688	1.771	26.537
100	1500				1.091	.376	5.885			
100	2000				1.113	.377	6.645			
150	250							5.309	4.835	8.371
150	500				2.354	1.243	6.92	10.311	5.544	29.545
150	750							12.865	5.780	44.985
150	1000				3.121	1.276	15.162	14.370	5.898	55.508
150	1500				3.428	1.287	19.693			
150	2000				3.593	1.293	22.443			
200	250							8.806	7.853	10.112
200	500				4.259	2.824	10.136	20.706	11.469	40.243
200	750							27.533	12.674	63.773
200	1000	0.928	0.372	3.226	6.544	2.991	26.236	31.750	13.277	80.281
200	1500				7.551	3.047	36.022			
200	2000	1.048	0.375	5.064	8.112	3.074	42.209			
200	3000	1.091	0.376	5.885						
200	4000	1.113	0.377	6.345						
250	1000	1.576	0.726	5.099						
250	2000	1.926	0.737	9.778						
250	3000	2.060	0.741	12.148						
250	4000	2.130	0.743	13.540						
300	1000	2.354	1.243	6.920						
300	2000	3.121	1.276	15.162						
300	3000	3.428	1.287	19.693						
300	4000	3.593	1.293	22.443						

8.6 Correlation between Experimental Results and Proposed Semi Empirical Equation

Five of the six arches tested in the laboratory fit the span to rise ratio for the optimum arch shape range used for the derivation of the generic equation. Their parameters are shown in Tables 8.8 and 8.9. The experimentally measured stiffnesses, the stiffness values from the proposed Equation 8.13, Table 8.7 (figures in red), and the stiffness calculated from elastic analysis, Table 8.1 (figures in red), are compared in Tables 8.8 and 8.9. The experimental stiffnesses are shown in Chapter 7 but are modified here to make the test arch width directly comparable with a 1000mm wide arch, used for the semi-empirical model, which in turn is reduced to 750mm by removing the effects of the butt joints. The modifications are as follows:-

- Active depth of laminate is the minimum solid depth of arch ring where laminates are in contact. This is less than the depth of the laminate because the laminates are not cut on the curve.
- For the 2.1m span arches settlement deflection was calculated from the average thrust to settlement ratio derived from the 6m span tests which used the same lateral restraint system. This varied from approximately 2.5 to 4.5 kN for 1mm of settlement. This modification was necessary because the settlement was not measured for the 2.1m spans.
- The geometry ratio is the proportion of vertical deflection resulting from a corresponding lateral settlement when the arch ring remains the same curve length. The spreadsheet shown in Appendix 10 is used to calculate this accurately.
- The 'deflection zero settlement' is the vertical experimental measured settlement less the 'geometric deflection' where lateral settlement has taken place. This is the true structural settlement for comparison with the proposed equations which were derived from arches where no lateral settlement took place.
- The 'K_{285}' is the stiffness of the experimental 6m span arches which were 285mm wide. 'K_{140}' is the stiffness of the experimental 2.1m span arches which were 140mm wide.

- The 'K_{750}' is the stiffness for the equivalent 750mm wide arches which is the width used in the derivation of the proposed equation. 1000mm arches were considered with 25% of the cross section removed to account for the butt joints.
- '$K_{proposed}$' is the stiffness calculated from the parametric equation, Table 8.7.
- K_{QSE} was calculated using the same 20kN of loading, applied as two third point loads, that was used in the test. The active depth of section was used to calculate properties. These values are taken from Table 8.1 for the 6m span arch (shown in red).
- 'Vertical deflection error' is the difference between the measured experimental deflection and the proposed equation.

The comparison of results, in Tables 8.8, 8.9 and 8.10, show some differences which result in quite large percentage errors in stiffness. These differences are related to small deflection readings which are within acceptable experimental error.

Table 8.8 – Stiffness comparison 2.1m span arches

Experimental tests, proposed equation and elastic analysis

2.1m Spans - Comparison of Stiffness						
Experimental K		Transverse Tensions			Full Tension	
		0 kN	3.71 kN	8.21 kN	12.64 kN	Modified
Arch Rise	335	1.75	4.37	4.48	4.36	85
	447	1.76	3.74	3.63	4.2	95
QSE Analysis K						
Arch Rise	335					91
	447					119
Proposed Equation K						
Arch Rise	335					63
	447					70

For the 6m span comparison, the values for QSE stiffnesses in Table 8.9 are taken from Table 8.1 (figures in red) and for the proposed equation, from Table 8.7 (figures in red).

The reduced stiffnesses include the effect of deflection due to lateral settlement and the semi empirical model values (proposed equation K) are calculated using Equation 8.13. The stiffness without settlement is the load which causes 1mm of structural deflection. With settlement that load effectively causes that 1mm plus the deflection due to settlement. Therefore the reduced stiffness is the zero settlement stiffness divided by:

$$1\text{mm} + \text{settlement deflection mm}$$

Table 8.9 – Stiffness comparison 6m span arch

Experimental tests, proposed equation and elastic analysis

6m Spans - Comparison of Stiffness									
Experimental K		Transverse Tensions			Full Tension		Settlement		
		0 kN	7 kN	12 kN	24 kN	Modified	1mm	2mm	3mm
	250	0.148	0.182	0.186	0.183	1.97			
	500	0.602	0.961	0.98	0.98	9.32			
	1000	0.135	0.974	1.56	1.764	5.86			
QSE Analysis K									
	250					5.076	2.3	1.52	0.9
	500					13.89	5.34	3.29	1.86
	1000					23.81	10.11	6.38	3.65
Proposed Equation K									
	250					5.7	1.01	0.56	0.29
	500					15.9	4.76	2.8	1.53
	1000					26.5	12.42	8.11	4.79

Table 8.10 – Stiffness comparison 20m span arch
Experimental tests, proposed equation and elastic analysis

20m Span – Comparison of Stiffness		
	K kN/mm Central Load	K kN/mm Quarter Point Load
Experimental	1.5845	0.7838
QSE Analysis	0.926	0.61
Proposed Equation	1.022	0.748

The largest error, between experimental results and the proposed equation, is in the 1m rise, 6m span arch. This could have been the result of the laminates slipping until the butt joints made contact creating an extra 6.7mm of deflection. This has been found to happen when constructing full scale arches. The preload in the laboratory, to ensure this would not happen, was 3kN therefore it is reasonable to assume that the 20kN test load cause some further slip.

The modulus of elasticity of the timber used for the experimental 6m arches was evaluated experimentally and is detailed in Sections 7.8 of Chapter 7. This was found to be 11.5kN/mm^2 which is greater than the value 8.8kN/mm^2, used for the parametric calculations and the proposed equation.

The deflection comparisons were based on the four point loading which was used in the laboratory. The results are within the range of experimental error so it is reasonable to assume that the proposed generic equation, for the other loading regimes, will give acceptable results.

A full comparison between the parametric study and the field tested arches is not possible because the lateral thrust and lateral deflections were not measured in all cases. However correlation has been shown between deflections and elastic analysis for these arches (6m trial arch and 15m test arch) and excellent correlation is shown in Table 10 for the 20m span arch where the lateral settlement was indicated by cracks in the soil at the bearings. This results is particularly significant and therefore it is reasonable to assume that the proposed equation will hold true for field tested arches.

8.7 Conclusions from Development of Semi Empirical Model

The parametric study has provided a set of equations, one for each of the critical loading regimes for arch bridges. Correlation, within experimental error, has been demonstrated with laboratory tests which performed in a similar way to the field tested arches. It can therefore be assumed that the proposed equation will give useful design information for all combinations of parameters within a span to rise ratio of between five and twenty four. It is significant that a peak value of stiffness, and thus strength, was found to occur at a span to depth ratio of approximately five for all arch shapes. This will be useful in choosing profiles for specific arch designs.

It is significant that the lateral settlement affects the vertical deflection but, due to the low modulus of elasticity of timber and the stress relief capability of SLT, settlement has little effect on the stress set up within the arch provided settlements are relatively small. This provides a considerable advantage over masonry arches where even quite small settlements can result in instability. This effect will eventually translate into lower cost foundations when the limits of allowable settlement are evaluated.

CHAPTER 9

9 CONCLUSIONS AND RECOMMENDATION FOR FUTURE WORK

The construction of low cost sustainable bridges is a worthy aim. The world is entering a period where specialist materials are in short supply, energy costs are escalating and atmospheric pollution is getting out of control. In this context anything that can be done to help the environment is politically, socially and morally justified. Throughout the world, bridge construction uses a significant amount of material and much of it requires energy-intensive processes to make it, transport it and build with it. As population rises, demands on the available materials increase and therefore costs escalate.

Timber is plentiful, carbon neutral, easy to work with and usually available near to the sites of bridges. Because plantation timber is now available from so many countries, the price will remain internationally competitive. Further, because it does not require great investment to establish plantations, it is possible that even developing nations can have a plentiful and affordable supply in the future.

Timber, therefore, has many advantages over steel and concrete but it has gone out of fashion as a primary building material. Structures have become political statements which demand new technology and hybrid materials. And so, significant research into novel methods to allow it to compete is needed. This is especially difficult today because most of the large trees were felled many years ago and plantation timber nowadays tends to consist of fast growing varieties and is, therefore, of small diameter.

Although there are significant technical, social, political and environmental reasons for building timber structures, more importantly, there is a market. The text of this thesis refers to similar practice around the developed world over the last thirty years of investing less on maintenance of minor roads. This neglect has taken it toll on the distributor and feeder networks which link rural production to the main highways. Many of these roads have bridges, from one hundred years ago, made of timber, particularly in

Australia and the USA. This backlog of maintenance promises to become a significant market over the coming years.

The Forestry Commission (FC) in the UK has always had to build low cost bridges to keep costs down so that the plantation timber can compete in the international market. For that reason, the FC has more experience of low cost bridges than other Authorities in the UK. Timber structures are of special importance to the FC because timber is its product and, being a Government body, it must support the political mood and policies. The FC was therefore very supportive of a new innovation using timber for bridges, especially as the results may be appropriate for use on minor public highways. This, therefore, is the background to the reasons for this PhD research and why this thesis began with a full description of what had gone before in the FC.

Glue lamination or LVL would have been a way forward but there are no existing facilities on which to build a new industry in the UK. That form of Timber Engineering also requires very good quality, highly machined timber, not readily available in the UK. Stress lamination was therefore an obvious candidate for research.

9.1 Main Conclusions

The important findings of this research are reported in Sections 7.9 and 8.7. The following summarises them:-

- Stresses in timber arches can be predicted reasonably accurately by elastic analysis.
- Simple elastic analysis cannot take account of slip and lateral tension forces.
- Load carrying capacity was beyond predictions.
- Timber arches are lightweight and take bending unlike masonry.
- Timber arches have a very high strength to weight ratio.
- Timber arches are stable even at relatively flat profiles.
- Internal slip relieves stresses.
- Only 1/3 of the initial prestress is required for full structural stiffness.
- Stressing bars will lock the structure before collapse could take place.

- SLT arches are significantly stronger and safer than SLT flat decks.
- SLT arches transfer load through short laminate laps unlike flat decks
- Lateral settlement has a significant effect on load deformation behaviour and therefore on stiffness.
- Tied arches save significantly on cost.
- Maintenance regimes to ensure safe lateral tension are vital
- Very flat slender arches are only restricted in their capacity by their FNF.
- Steeper arches are stronger but there is a limit at span/rise = 5.
- Future designs can make use of the semi empirical model.
- Environmental credentials of the system are good.

Collectively these findings have significantly advanced the understanding of timber stress lamination techniques. Arch construction of this type has not knowingly been used before, except by Philip De Lorme in the 16th century, and he did not have the advantage of hydraulic jacks and prestressing steel. Arches now seem to be a logical extension to the recent work on flat slabs and opens up many possibilities for other timber structures.

The results of this work are likely to have significant commercial value to Authorities who require bridges and also to timber producers with stocks of plantation softwood.

9.2 Future Work

The future use of stress laminated arches is most likely to be in recreation structures, vehicle bridges, roofs and floors for buildings, therefore future research work should be focused on the requirements for these uses. It will provide low cost spanning using compression which is one of timber's better qualities. There are many possibilities e.g. domes and full arch buildings. Because timber of the quality required will be available for between £300 and £400 per m^3 for the foreseeable future, long term research should be justified.

Work needs to be done on the optimum amount of settlement which will occur and which can be tolerated. Pre-cambers will be built in to negate the dead load settlement.

The fire resistance of the timber slab needs to be investigated. It is likely that it will act like very large timbers and char, thus creating an insulation layer against further combustion. The small spaces between some laminates may negate this, so tests need to be carried out. This will be an important precursor to use for roof and floors for buildings.

Successful heavy vehicle bridges will employ an arch supporting a flat stress laminated deck. Before this can be done successfully, some research is necessary to optimise the joints between the arch and the deck. If rigid steel joints are used they may induce high stress points and fatigue whereas, if an integrated timber jointing system is developed, the entire system would have similar modulus and avoid high stress points.

Preservative treatment is not only expensive but it reduces the environmental value of these structures. Work needs to be done on the economics of using the most durable timber available e.g. Larch or Douglas Fir, without treatment. The economics would allow for replacement at approximately fifteen years. This may seem a short life but there will be no maintenance costs, very low capital costs and for small enough bridges, the decks can be prefabricated and changed over very quickly. Very vulnerable areas could be selectively treated, in situ.

The stiffening effect of the bitumen surfacing needs to be evaluated more accurately. The thickening at the end of the bridges helps in this respect but it is mainly the stiffening effect of the bonded layer which helps increase FNF.

More work is required on stressing systems for specific uses. It may be cost effective to use carbon rods, or similar, for buildings because these materials do not relax after time, as steel does. It may also be cost effective to use much lower cost threaded bar for recreation structures and thus avoid the requirement for stressing jacks.

Finally, to further the understanding of the action of a stress laminated timber arch, a plan area of approximately $1m^2$ in the centre of a deck needs to be analysed in detail, to provide a true inertia and modulus for the deck. This has not been done elsewhere for a timber arch.

The foundations have required large quantities of concrete which significantly reduces the sustainability of the system. Research is required to evaluate the passive resistance which can be provided by soil in the horizontal direction. Foundations could then be designed with the use of very much smaller quantities of concrete.

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APPENDICES

APPENDIX 1

STRESS LAMINATED BRIDGES

AERIAL MAST BRIDGES

SPECIFICATION FOR SOFTWOOD TIMBER DECK MEMBERS

……………. BRIDGES FCE/

All timber must be sourced from a sustainably managed UK forest registered under the Forestry Stewardship Council or with equivalent credentials. This will have to be proved by the supplier either with signed documentation or proof of membership.

Species	Home Grown European Larch, Corsican Pine, Scots Pine, Douglas Fir
Strength Class	C16, C24 to BS 5268: Pt 2 1996. Timber to be visually stress graded in accordance with parameters given below in Forestry Commission Stress Grading of Bridge Timbers.
Action before Treatment	Cut to size, Drill holes according to schedule. Kiln Dry to moisture content 12%.
Preservative Treatment	Copper Chromium Arsenic (CCA) 5.3kg/m^3. For Deck members Copper Chromium Phosphate (CCP) For Handrail Members
Post Treatment Drying	Dry to 18% moisture content (CCA). Allow 2 weeks for CCA to chemically lock. Seal end grain of deck timbers.

APPENDIX 2
FORESTRY COMMISSION BRIDGES
STRESS GRADING OF SOFTWOOD TIMBERS

The following specification will be used to identify suitable pieces of home grown softwood timber which have not been machine graded or visually stress graded to BS 4978. This identification technique is ultimately a short cut visual grading which is relevant for timbers used for these particular bridge decks and specified upto C24.

<u>Permitted Timber Species</u> — <u>Species</u>
Pine, Larch, Douglas Fir

<u>Rate of Growth</u> — Average width of annual rings no greater than 6mm for C24 and up to 10mm for C16

<u>Fissures</u> (Resin pockets and bark pockets)
Codes permit up to ½ way through the thickness anywhere and all the way through for a restricted length. This specification takes a conservative approach and allows only up to 25mm deep splits for a 500mm length.

<u>Slope of Grain</u> — Not steeper than a gradient of 1 in 6.

<u>Growth Ring Distribution</u> — No fully boxed heart permitted

<u>Wane</u> — ¼ of width for full length of a timber is acceptable or 1/3 of the width for a distance of 300mm.

<u>Knots</u> — Knot area at any cross section not greater than 1/5 of total cross sectional area.

<u>Distortion</u>	Bow	-	Maximum 10mm over 2m	[ends up]
	Spring	-	Maximum 5mm over 2m	[sides out]
	Twist	-	Maximum 8mm over 2m	[corners up]
	Cup	-	Maximum 6mm	[bow across section]

<u>Worm Holes</u> There shall be no wasp holes but pin worm holes are permitted in small numbers which will not affect the wood strength.

<u>Fungal Decay</u> Reject any piece with any decay.

<u>Sapstain</u> This is not a defect.

<u>Abnormal Defects</u> Abnormal special defects eg compression wood, which would weaken the plank below its serviceability are grounds for rejection.

APPENDIX 3

Determination of Mechanical Properties of Timbers for Laboratory Bridges

Determination of density:

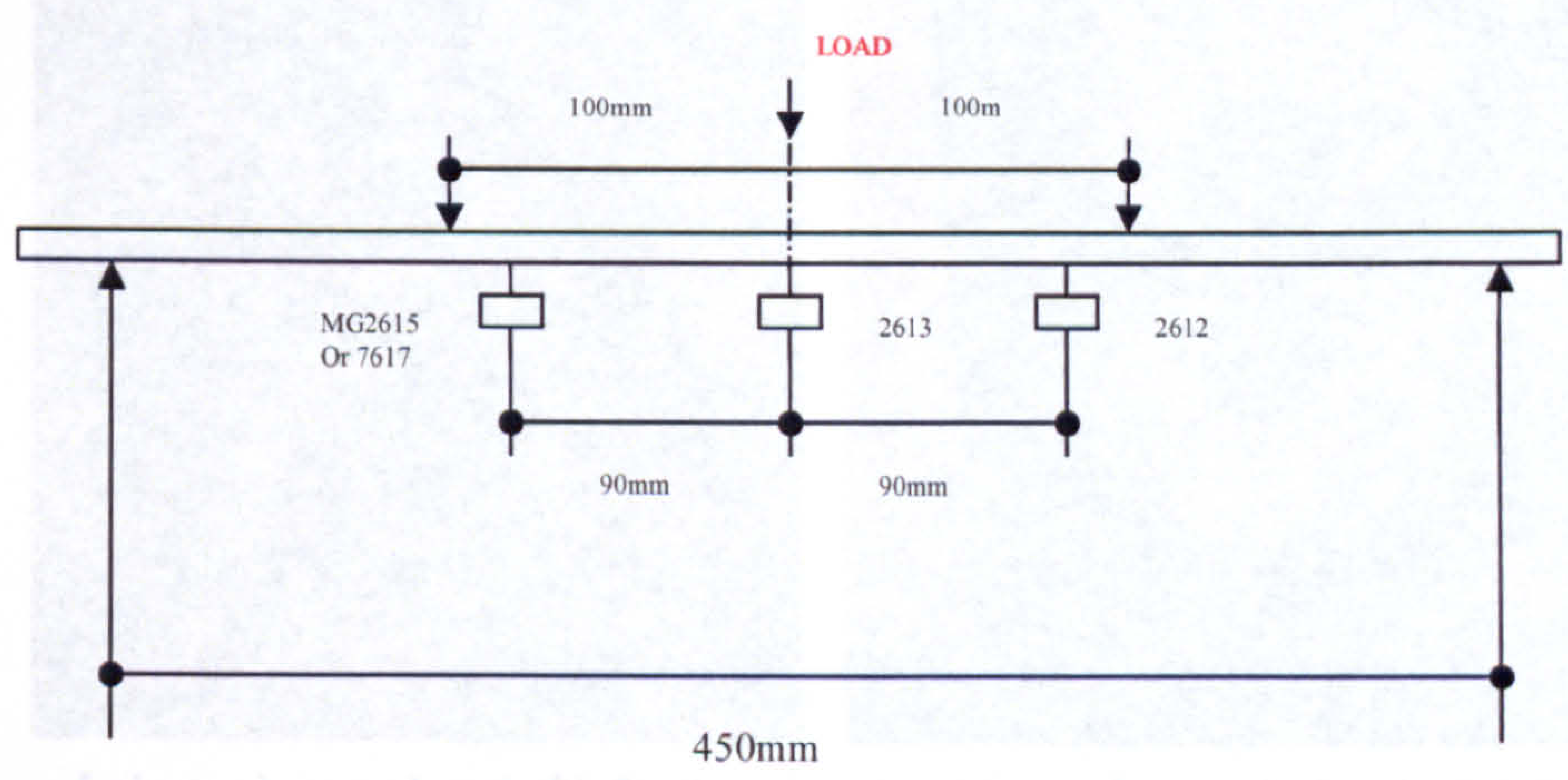
Density of the test pieces was determined by cutting three sections from the samples used for static bending. In order to provide a density value for the wood used in all tests an average was taken of these three samples.

Determination of moisture content

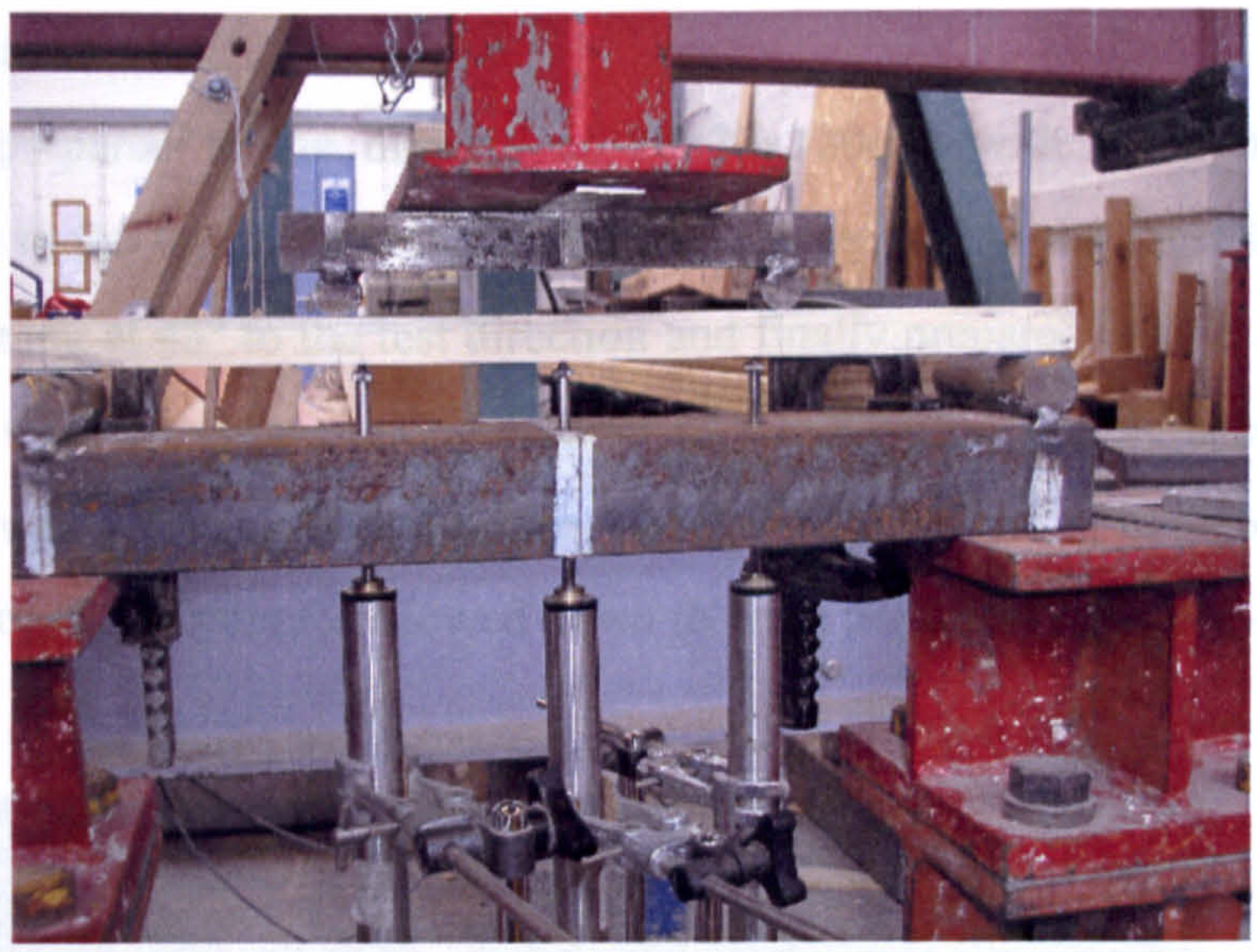
Moisture content was determined on the same samples that were used to measure density. Mass was measured after the test, the samples were then placed in an oven >100°C until such time as they were deemed to be of constant mass. Moisture content was then calculated as (wet mass – dry mass)/dry mass × 100.

Static Bending

Three samples were prepared to the dimensions 20×20×380mm. Dimensions were checked prior to testing with Mituyoto Callipers. The test machine was a Zwick Rowell with 100kN load cell set up with three transducers as shown in figure one and two. The bending speed was 0.5mm per second. Transducers were removed after a load of 1000N in order to prevent damage to the equipment. Modulus of Elasticity was calculated according to BS EN 408.



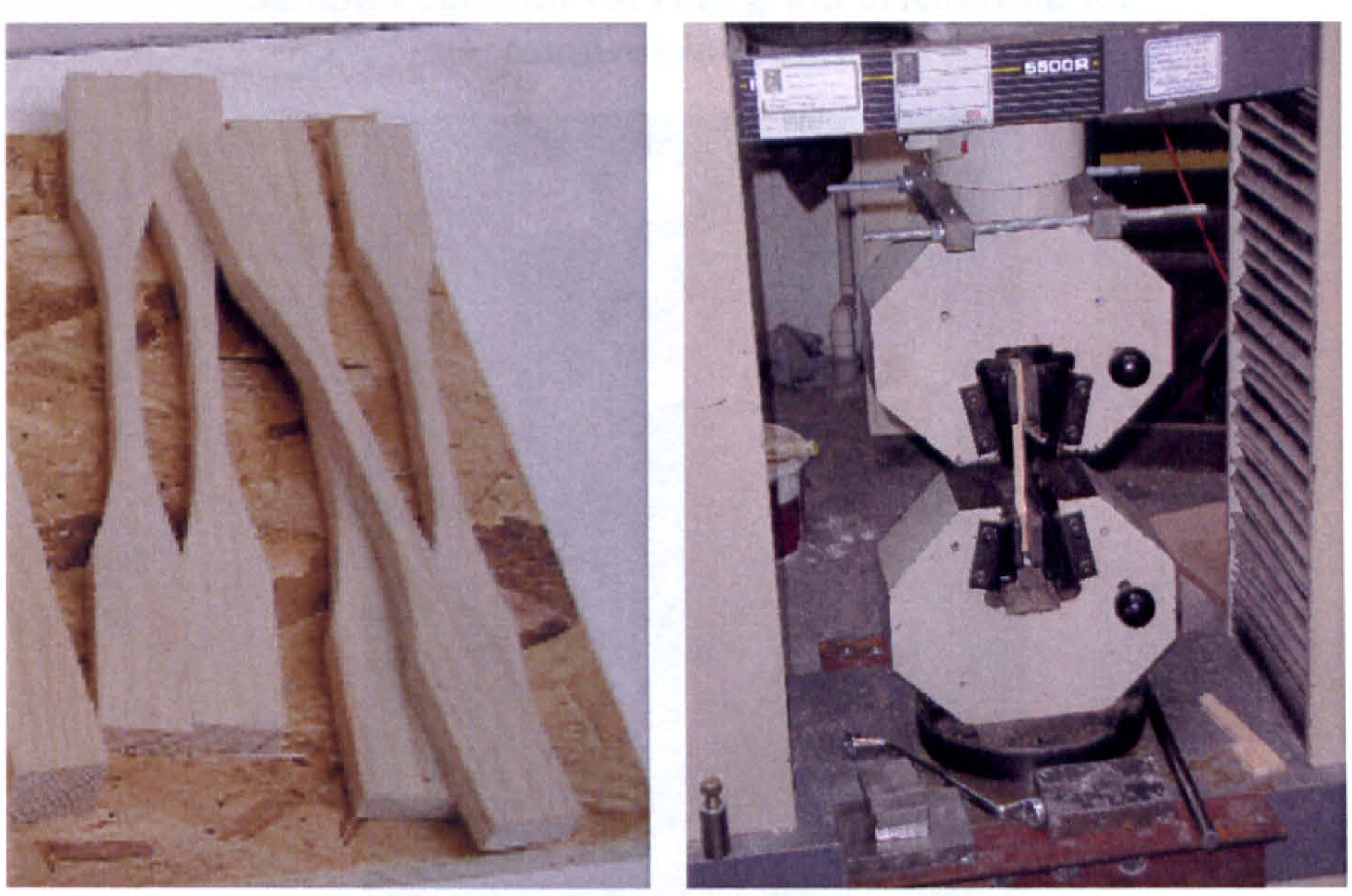
Set up for static bending test



Static Bending

Tension Test

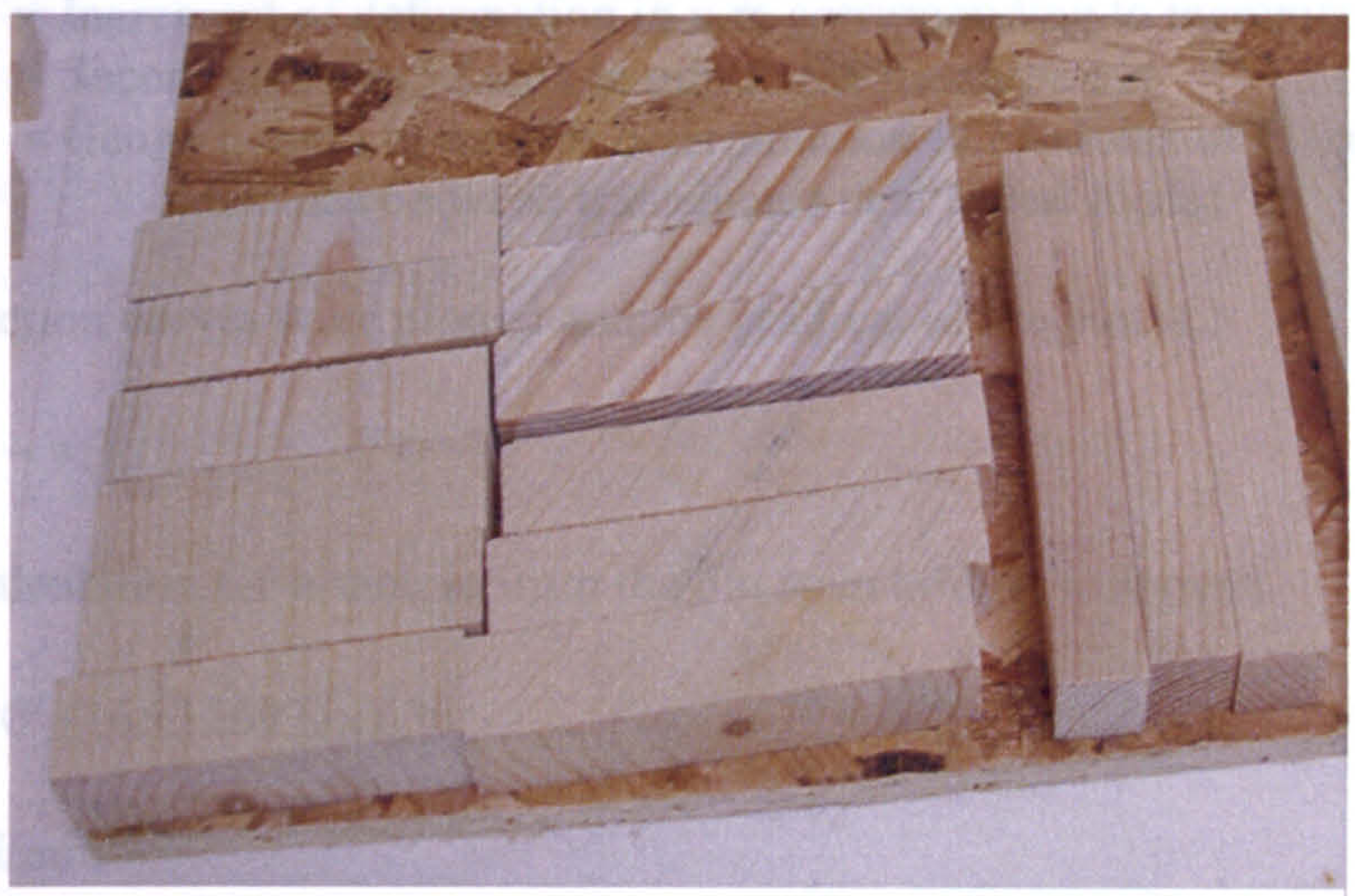
Three samples were prepared in an 'I' shape. The test machine was a Schenk Trebel with a 100kN load cell, rate of tension was 0.5mm per minute, speeded up to 3mm per minute. BS EN 408 states that a tension speed of no more than 0.15mm per minute should be used, however due to constraints on time this was not possible. Modulus of elasticity was determined according to BS EN 408:



I shaped samples (left) for tension testing and sample in rig (right)

Compression Test

3 samples were prepared to the dimensions 20×20×120mm with the grain running perpendicular (0°) to the test direction, 3 samples were prepared to the same dimensions with the grain running at 45° to the test direction and finally prepared were 3 samples to those dimensions with the grain running parallel to the direction of compression (90°). The test machine was a Schenk Trebel with a 100kN load cell, rate of compression was 3mm per minute for the 0° samples, slowed down to 1mm per minute for the 45° and 90° samples.



Samples cut with varying grain orientation for compression testing

Compression Testing

Calculation in accordance with BS 08
Modulus of Elasticity in Bending:

$$E_m = \frac{a l_1^2 (F_2 - F_1)}{16I(w_2 - w_1)}$$

E_m	= Elastic Modulus
a	= distance between loading position and the nearest support mm
$F_2 - F_1$	= Increment of load in straight line portion of graph in N
$w_2 - w_1$	= Increment of deformation in mm corresponding to increment of load
I	= Second moment of area in mm to the fourth power
l_1	= Gauge Length in mm for determination of Modulus of Elasticity (distance between transducers next to test loads)

The load deflection curves were plotted and the gradient of the plot is :

$(F_2 - F_1)/(w_2 - w_1)$

An example calculation for a sample from the 6m span bridges is shown:

$E_m = (125 \times 180^2 / 16 \times 13697) \times 636$

$E_m = 12068\ \text{N/mm}^2$

Results

The average density = 547kg/m^3 assumed 500kg/m^3

The average moisture content = 10.8% assumed < 18%

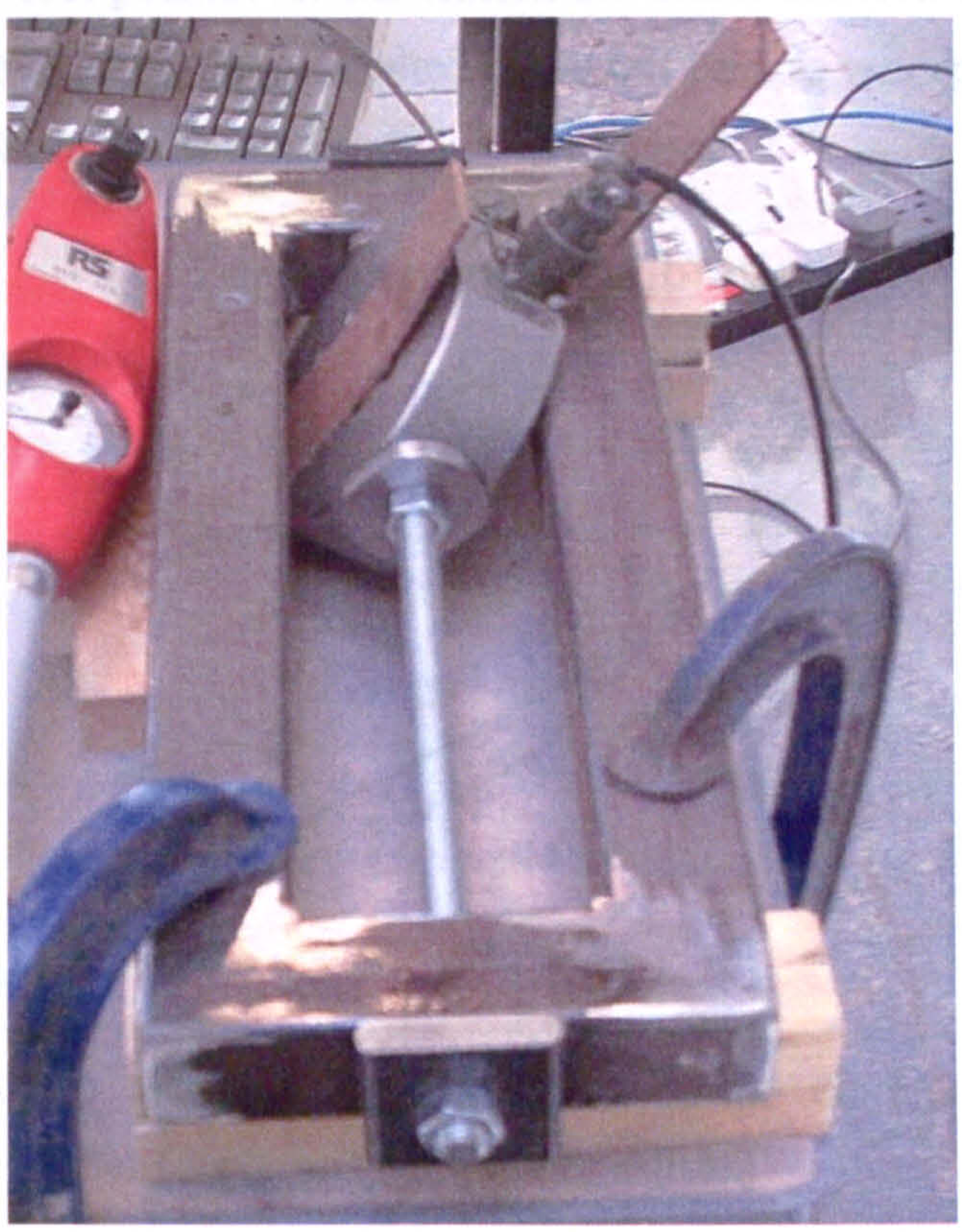
The average modulus of elasticity 6m span = $12642\ \text{N/mm}^2$
2.1m span = 4590N/mm^2
assumed 8800N/mm^2

APPENDIX 4

Torque wrench calibration for mild steel threaded bar

To induce a specific tension in the threaded tension rod a torque wrench was used to tightening up the nuts. By carefully using the wrench it was possible to determine the specific tension in the bars in the bridge structure. To find the torque which created the required tension, the torque wrench had to be calibrated.

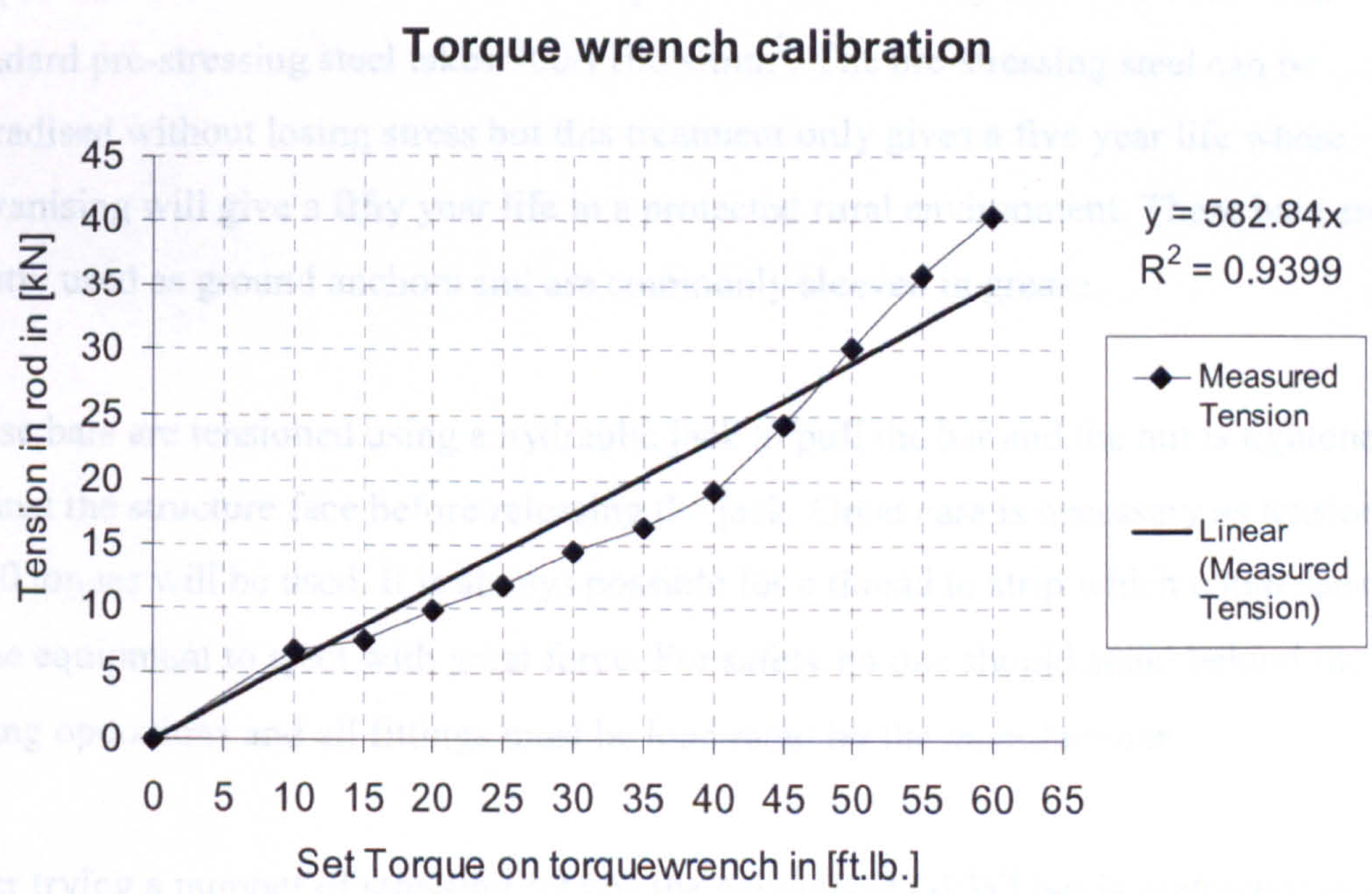
This was done by means of a steel frame, a load cell and a short piece of the threaded tension rod. The load cell was fixed inside at one end of the frame. A short piece of the tension rod was fixed to the load cell at one end while the other passed through a hole in the frame to the outside. There the stressing bar was fixed with a washer and a nut.



Torque wrench calibration setup

Turning the nut by means of the torque wrench tensioned the threaded rod. This tension was measured by the load cell. The torque wrench could be set to a specific torque that was indicated with a snap when reached. The tension, that equated to the torque was measured by the load cell and recorded on computer. Each torque was measured seven times in the range from 35 to 55 foot. This was necessary to average out the errors resulting from strain on the threads with increasing torque. At the end of the calibration it was impossible to remove the nut from the rod because the threads had deformed too much. To minimise this affect the threads were then treated with a large amount of WD40 to preserve them by reducing the friction.

The upper limit for the tension was found by torquing against a short a piece of timber which had been used for the bridge constructed. This test gave a maximum torque of approximately 65 foot pounds. At this tension the timber suffered bearing failure under the washer. The maximum torque applied to the test bridges was approximately 44 foot pounds. At this torque the washer bearing failure began.



APPENDIX 5

Dywidag and Gewi bar Characteristics

Besides the threaded bar used in the laboratory, two types of stressing bar were used in the project - standard pre-stressing steel and galvanised stressing steel, by 'GEWI' from Germany.

Dywidag pre-stressing steel is easily available and is generally used in the construction industry for holding temporary shutters for concrete. They have a very high tensile stress capacity and a very high pitch of thread which makes them ideal where strength and holding capacity is required. They are no good for tightening up in the way a bolt is used because of the type of thread. However this type of thread is very much stronger and less easily damaged.

These stressing bars are made by deforming hot rolled steel and, if galvanised, the heat in the process causes stress loss. This is why the GEWI can only take 500/600N/mm^2 where standard pre-stressing steel takes 900/1100N/mm^2. The pre-stressing steel can be sheradised without losing stress but this treatment only gives a five year life where galvanising will give a fifty year life in a protected rural environment. These bars are mostly used as ground anchors and are commonly sleeved in grease.

These bars are tensioned using a hydraulic jack to pull the bar and the nut is tightened up against the structure face before releasing the jack. Great care is necessary as tensions up to 20 tonnes will be used. It is always possible for a thread to strip which could cause some equipment to eject with great force. For safety no one should stand behind the jack during operations and all fittings must be load rated by the manufacturer.

After trying a number of stressing set ups the galvanised GEWI bar is preferred because it is simple to use and is galvanised. The next bridges will try stainless steel to compare costs, availability and effectiveness. A typical set of GEWI or pre-stressing bars, nuts and washers for a 20m bridge will cost between £1,500 and £2,000.

Prestressing Steel Threadbars

Based on a continuously threaded bar, the Dywidag System of Prestressing Steel Threadbars is exceptionally versatile and well proven worldwide.

The threadbars may be cut to finished length at the factory or on site and anchorages, nuts or couplers fitted immediately without frustration or delay.

Being coarse and robust, the thread is ideally suited for use in the construction industry.

For CS Timber use stress to 50% yield/ultimate ie this will be 10t for 15mm & 20 for 20mm.

Technical Data for Prestressing Steel

Bar Diameter mm	Steel Grade Yield/Ultimate N/mm²	Ultimate Strength kN	Yield Strength kN	70% Ultimate kN	Weight kg/m	Cross Section mm²	Pitch mm
* 15	900/1100	195	159	136	1.44	177	10
* 20	900/1100	345	283	241	2.56	314	10
* 26.5	900/1030	568	496	398	4.48	551	13
26.5	1080/1230	678	595	474	4.48	551	13
* 32	900/1030	828	724	579	6.53	804	16
32	1080/1230	989	868	692	6.53	804	16
* 36	900/1030	1049	916	734	8.27	1018	18
* 36	1080/1230	1252	1099	876	8.27	1018	18
* *Available from Stock*		Modulus of Elasticity. E = 205,000 N/mm² ± 5%					

Anchor Plates

Solid Flat Recessed Anchor Plate and Domed Nut

When anchoring prestressing steels the Dywidag nut is usually domed shaped. This nut locates into a cone shaped recess in the anchor plate and can tolerate a nominal deviation from the Normal. The flat anchor plate is designed for surface mounting on either concrete or steel. Variations on plate size are possible from the standard table to suit specific applications. Grouting holes may be incorporated to suit.

Bell Anchorage with Domed Nut

The bursting forces immediately behind this anchorage are contained by a steel cylinder. This causes a triaxial stress condition resulting in a uniform load transfer. Incorporation at design stage is essential as bell anchorages must be cast in situ.

Articulating Anchor Plate

This anchorage comprises a cast steel base plate and a malleable cast iron hemisphere. The hemisphere is designed to accept all preferred sizes of domed anchor nuts. The hemisphere permits articulation of up to 30° in one direction. Applications include tie bars through structures or piling where tendon orientation cannot be predicted.

GEWI® Steel High Yield Threadbar System

GEWI® Steel High Yield Threadbar is a high tensile alloy steel bar which features a coarse left-hand thread over its full length. The system is proven worldwide and offers versatility in a range of applications.

Manufactured in accordance with the German Certificate of Approval (Deutsches Institut für Bautechnik), the system also offers general conformance with BS 4449 : 1997 (Carbon Steel Bars for Prestressing of Concrete).

The minimum specified characteristic yield strength is 500 N/mm^2 for bar diameters 16 - 50mm and 555 N/mm^2 for the 63.5mm diameter bar. 16 - 50mm bars can be welded using appropriate industry practices relative to the carbon content of the steel. Welding of the higher grade 63.5mm diameter bar is not recommended.

Key features of the system are:

- Fully Threaded Bar - can be cut and coupled at any point.
- Robust Threadform - ideal for construction site use.
- Coarse Pitch Threadform with two flats – ensures thread is self cleaning.
- Fully Galvanized Systems - galvanized threadbars and accessories also available from stock.

Technical Data for GEWI® Steel High Yield Threadbar

Nominal Diameter	Steel Grade	Ultimate Strength *f*pu	0.1% (a) Proof Stress	70% (b) Ultimate Strength	50% Ultimate Strength	Cross Sectional Area	Diameter Over Threads	Thread Pitch	Bar Weight
mm	N/mm^2	kN	kN	kN	kN	mm^2	mm	mm	kg/m
16	500/600	121	100	85	61	201	19	8	1.58
20	500/600	188	157	132	94	314	23	10	2.47
25	500/600	295	245	206	147	491	29	12.5	3.85
28	500/600	370	308	259	185	616	32	14	4.83
32	500/600	482	402	337	241	804	36	16	6.31
40	500/600	756	630	529	378	1260	45	20	9.87
50	500/600	1176	980	823	588	1960	56	26	15.40
63.5	555/700	2217	1758	1552	1108	3167	69	21	24.80

(a) 0.1% Proof Stress also referred to, in general terms, as Yield Strength - Ty.
(b) For geotechnical applications 75% fpu may be used for proof testing.

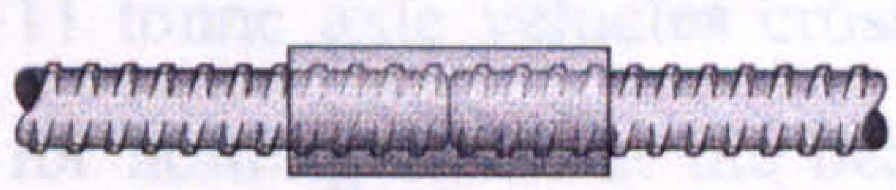
Modulus of Elasticity: E = 205,000 N/mm^2 +/- 5%.
Stock Lengths: All bar diameters 12.0m. Tolerances +/- 100mm. Special lengths up to 18.0m are available to order.
All bar diameters can be cut to length to suit customer requirements or supplied bent to BS 8666 : 2000.

Couplers for GEWI® Steel High Yield Threadbars

Couplers enable GEWI® Steel Threadbars to be coupled or extended, reliably and efficiently. It is important that the two threadbars meet centrally within the coupler, and remain so during installation, to ensure correct load transfer. Coupler strength is 92% *f*pu of the bar in accordance with the German Certificate of Approval.

With all types of coupler, precautions should be taken to ensure that the coupler remains centrally located during handling and installation. This can be achieved through the use of grub screws, a centre pin or lock nuts.

The choice of coupler type depends on the application. The static coupler is used either in constant tension or a combination of tension and compression loading, with the longer dynamic coupler used when vibration and cyclical load reversals are anticipated.


standard coupling


centre pin - optional


paint - visual indicator


grub screws - optional

Lock nuts should be used at each end of the coupler when the threadbar is used in reinforced concrete applications designed in accordance with BS 8110. These lock nuts should be torqued to a predetermined value to prevent cracking of the structural concrete at the coupler when the joint comes under load. See table on Page 3 for torque values.

APPENDIX 6

Bridges in Australia, USA and Scandinavia

Australia

In 2003 the Author visited the University of Technology in Sydney to witness the work carried out by Professor Crews [48] on Mechanical Stress Lamination. Crews had taken up the study after a visit by Ritter, M.A.[40] who had developed the subject in the USA. Crews advanced the developments of the flat slab, flat cellular decks by carrying out rigorous full scale tests which had not been done before. He carried out assimilation work on Australasian timber species for creep and loss of pre-stress.

The Crews research was carried out with funding from highway authorities with a view to constructing bridges on the main highways. Over a ten year period conclusive research results lead to the construction of a number of vehicle bridges which the Author visited with Professor Crews. The techniques for slab and cellular SLT construction are now well established in Australia where there is a very large market for these bridges. Australia has a good, dry, climate for timber bridge construction and there are a large number of roads with minimal traffic. These are perfect condition for SLT whereas, in the UK, with more severe conditions, there will be a demand for new details and designs to avoid premature failure.

The experience highlighted the possibilities for the UK. In Australia SLT decks are used on roads with speed limits of 70mph and the Author was able to witness the effects of fast moving 11 tonne axle vehicles crossing these bridges at high speed. The detail of the designs for holding down at the bearings, the topping, the maintenance schedules etc. were all of great interest and relevance to future UK practice. The UK is in need of many rural bridge replacements which could be achieved at low cost using SLT.

United States of America

In the USA from 1992 to 2002 the Author gave three papers at the American Society of Agricultural Engineers which houses the Forestry Engineering specialism. Contact was made with Professor Taylor [75] who has built many glue and stress laminated bridges in forest situations. It was this meeting which stimulated the first transfer of knowledge into the UK. This culminated in Prof. Taylor giving a paper on the subject in Edinburgh at an International conference, organised by the Author, in 1999. This stimulated interest and resulted in some funding.

The most important lesson to be learned from the USA is the durability of these structures. The modern construction of SLT has been used there for more than twenty five years. Prof. Taylor has carried out checks on long term loss of prestress which will be critical in the acceptance of this type of bridge for general use. The form of construction has been supported by a national initiative with funding and many States have responded by building SLT bridges. This level of confidence will be important in convincing UK Authorities to adopt SLT bridges.

Timber bridges were the norm for construction throughout the USA one hundred years ago and surprisingly some are still in very good condition today. The Author toured some of these covered bridges in Iowa in Madison County to find out why these bridges had lasted so long. The most common argument against timber bridges is their durability and information on very old surviving bridges is very useful.

Scandinavia

The Author led a DTI International Technology Mission to Sweden and Norway in 2000 to study the use of gravel roads for public roads in the UK and timber bridges for public roads. A previous contact from a conference in 1990, Otto Kleppe, from the Norwegian Public Roads Administration and past chairman of the Nordic Timber Council, escorted the Mission in Norway and introduced Erik Aasheim, Norwegian Institute of Wood Technology to the group. Between them they have lead a renaissance in timber bridge construction in Scandinavia. This has included a research programme in SLT decks and resulted in eight hundred new bridges in the region. The Mission group were shown some examples of the most recent construction which were extremely relevant to UK practice because the climate is more similar than in Australia.

In 1990, as part of a government initiative, Otto Kleppe toured the world on a fact finding mission to learn about good timber bridge construction. This lead to contact with Ritter, M.A. and stress lamination and its introduction to Scandinavia. It is used successfully in forest situation as well as on public roads where it is used to display the natural home grown material and build confidence in timber construction generally. Otto Kleppe took back one very important lesson from his tour. It was that the timber bridges which had lasted one hundred years had all been kept dry. They were the covered bridges in the USA and the Orient where the bridges had roofs for shedding snow or for decoration. However this is not possible with modern bridges because of the headroom required for large vehicles. He has therefore covered main members with metal copings and the timber is always treated with the most effective preservative allowed. He gives his bridges a one hundred year life which is very encouraging for timber bridge construction.

Pictures of Timber Bridge Construction from the Author's Tours.

The following pictures show a collection of bridges with a caption to explain what the picture is illustrating. The collection is limited to aspects which affect this research either directly as stress lamination or as examples of relevant timber bridge construction.

AUS - High speed road over SLT bridge on a bend with centrifugal loads

AUS - Extended bars for restressing – Continuous channel to spread concentrated load
Sufficient resistance can be achieved to support a crash barrier

AUS - Cracks at interface between timber deck and abutment resulting from deflection and no designed joint

AUS - Typical rural road where SLT bridges are best suited

AUS - SLT deck laid over failed existing deck and using existing piers and abutments

AUS - Typical round timber braced piers supporting SLT deck

AUS - Typical pier head detail

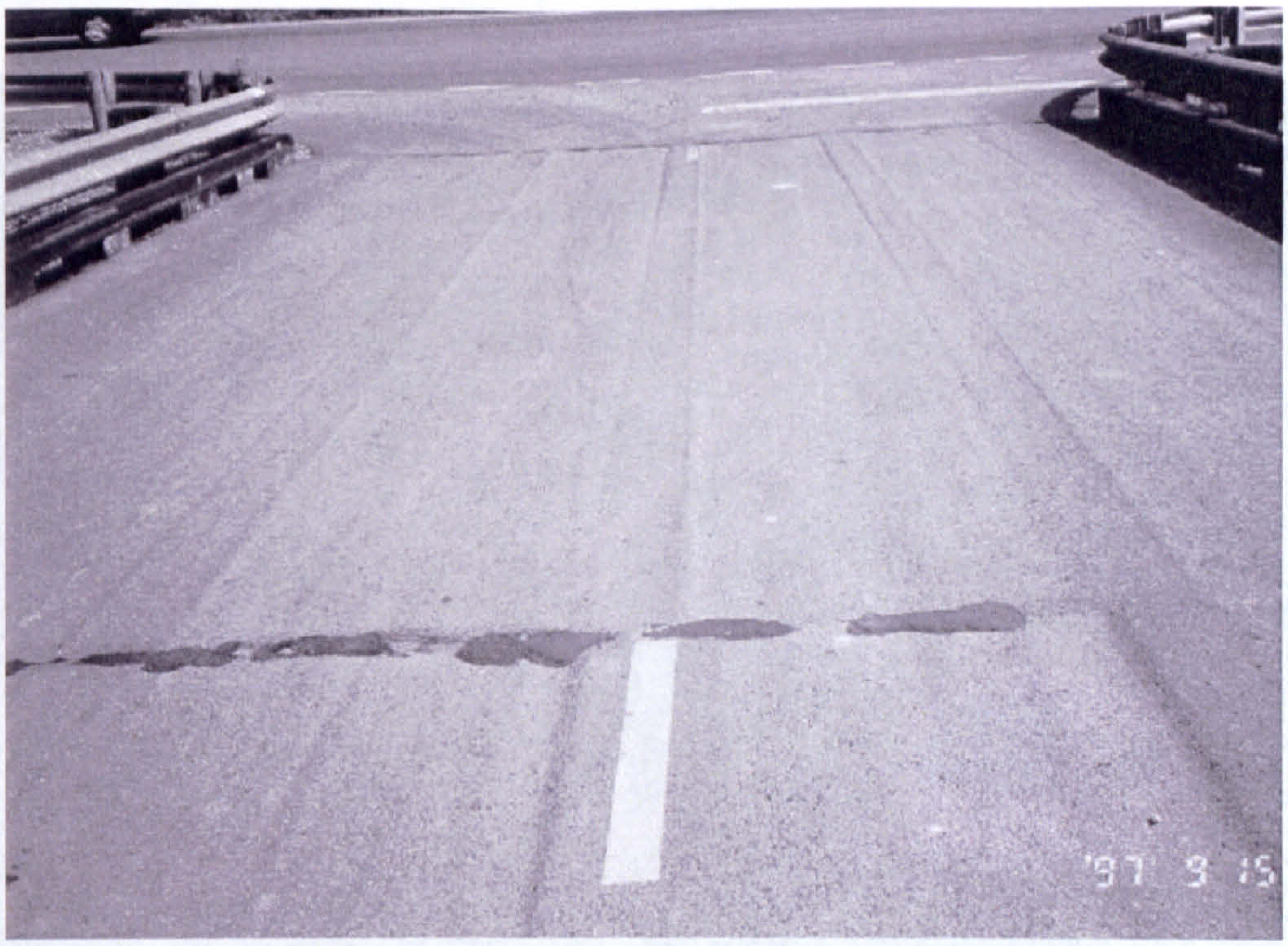
AUS - Metal plates to keep timber dry and tarred stressing bars to prevent corrosion

AUS - Typical bridge deck being replaced by SLT deck

USA – Typical cellular deck for full highway loading

USA – Typical "T" beam deck

USA – Restressing of SLT deck



USA – Cracks in asphalt at joints in deck and some slip of laminates

USA – 100 year old covered bridge in Madison County

USA – The internal bracing of the Roseman covered bridge in Madison County

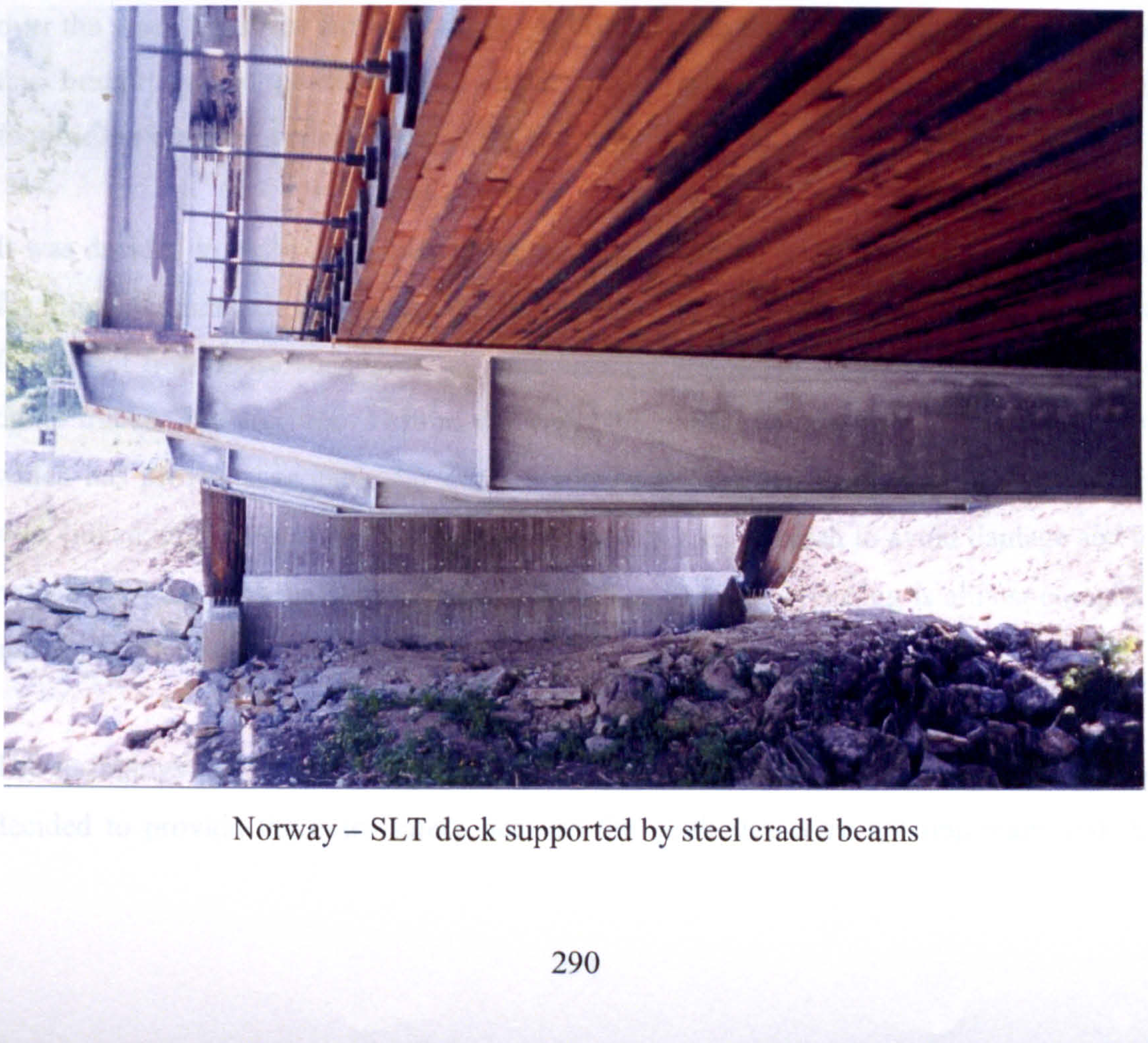
USA – Deck bracing of covered bridge

Sweden – Forest Road over a stress laminated timber flat deck

Norway – Stress laminated Laminated Veneer Lumber bridge on trunk road

Norway – Two timber bridges over two motorway carriages for minor traffic

Norway – Glue laminated timber arch supporting SLT deck on trunk road



Norway – SLT deck supported by steel cradle beams

APPENDIX 7

Case study of arch bridge for forwarders

Throughout the period of research a number of opportunities arose to build SLT bridges on a semi trial basis. That is, an actual bridge is required on the Forestry Commission Estate and because there are no Authorities to satisfy innovative structures can be trialled as long as they do not fail catastrophically. When a situation arises for a temporary bridge it is especially useful as even more inventive ideas can be tried.

A year ago in Spring 2005, a temporary forwarder bridge was required in Wales. These are vehicles, generally having 4 axles, for carrying timber from the harvesting site to a place for pick up by a road going lorry. They weigh up to 36 tonnes when loaded.

Forwarder bridges usually span short distances as forests in the UK tend to be in the uplands near the source of rivers. It is also useful if bridges are portable which means weight is important. Many types of portable or very temporary structures have been tried over the years but stress lamination has not been used in the UK. The particular remit for this bridge was to produce a forwarder bridge which could be converted to two footbridges after the working year of the harvesting site.

It was decided to build two 9m span arches each 1500mm wide. The arch was designed for the maximum axle load at the quarter point of the span and an allowance was made for impact from these very large off road vehicles which are very like construction site dump trucks. The arch was 175mm thick and the profile was based on the 1 to 12 ratio which has proved acceptable for other footbridges. The two bridges were laid side by side, linked with a steel bracket at mid span and covered in brash to avoid damage and to reduce the steep slope at the start of the arch. The year of harvesting is almost complete and the project has been a major success.

The springings were tied with steel ropes although the design did not require them. It was decided to provide the extra safety because the aesthetics were not important and the impact load is difficult to estimate. The performance of the bridge was monitored during use and it was estimated that it deflected 1mm under full load. The following pictures show the bridge in use, before and after harvesting :-

36 tonne harvester crossing on top of a brash mat

Before

After

APPENDIX 8 - LOADS FOR FOOTBRIDGES IN THE COUNTRYSIDE

MEMBER	LOAD TYPE	LOADING		REMARKS
		UDL	POINT LOAD	Use BS 5400
MAIN BEAMS	Pedestrian – Normal Crowd	2.3kN/m2 3.2kN/m2		for Urban or Wide bridges Use BS 5400
	Horse Cattle Sheep	3.2kN/m2		
	Quad Bikes		10kN – 20kN	
SHORT SPAN DECK BOARDS	Pedestrian – Normal Crowd Sheep		1.62kN on 75mm square	
	Horses & Rider		7kN on patch 300mm square	
	Cattle		6.12kN on patch 120mm square	
HORIZONTAL HANDRAILS	Pedestrians – Normal	0.74kN/m 1m above deck		
	Crowd	1.4kN/m 1m above deck		BS 5400 normal
	Horse & Rider Cattle	1.3kN/m 1.25m above deck		Handrail heights are not the same as height of load application
	Cycles	1.4kN/m 1m above deck		
	> 3m drop below bridge deck	1.4kN/m 1m above deck		
ALL MEMBERS	SNOW	0.4kN/m2		
	WIND	1.4kN/m2 loaded 0.7kN/m2 unloaded		Consult engineer for long spans
	COLLISION		50kN @ 3m high	

APPENDIX 9

Footbridge Component Design

Standard Foot Bridge - Beam Calculations SALCEY WALKWAYS

PROGRAMME DESIGNS ONE BEAM — Use Universal Mild Steel "I" Beams or rectangular timber beams with timber deck & rails

Inputs in RED — Moments in Green — Deflections in BLUE

SPAN =	9	m	Span of Bridge betw centreline of supports
Beam Spacing	0.7	m	
Deck edge Cantilever	0	m	
Thickness of deck	75	mm	
Timber deck density	7	kN/m3	
Timber Beam permissibl	7.5	N/mm2	
Timber Beam - Mean E	10800	N/mm2	E Steel = 210000N/mm2

Deflection Limits

Deflections are to be span/240 maximum

MAX DEFLECTION ALLOWED = 38 mm

Dead Loads

timber handrails	0.25	kN/m run	0.48 horse - 1m pedestrian 0.25
Timber deck/m2	0.60	kN/m2	15% extra added for runners
Beam Wt in kN	0.17	kN/m run	see below for weight under Beams
Edge DL	0.63		
Central DL	0.59		

Live Load UDL pedestrians in kN/m2 on the deck

Rural = 2.5kN/m2 Horses and Cattle = 5kN/m2

Crowd = 3.2kN/m2

FC crowd = 4.0kN/m2

Live Load kN/m = 3.2 kN/m

Full Wt ATV on 2 bearr = 10 kN

Wind - ignore up to 12m span 25% overstress takes it

Edge Beam Moments

DL Mmt edge beam =	6.4	kNm
LL Mmt edge beam =	11.3	kNm
TOTAL DL+LL =	17.7	kNm
TOTAL DL+ATV =	26.4	kNm

Mid Span Beam Moments

DL Mmt cent beam =	6.0	kNm
LL Mmt mid beam =	22.7	kNm
TOTAL DL+LL =	28.7	kNm
TOTAL DL+ATV =	26.0	kNm

LL Mmt ATV each Beam = 20.00 kNm

Timber Beams - permissible moment and deflections

				Deflections Edge Beam mm				Deflections Mid Beam mm			
	Z mm3	I mm4	Mr kNm	LL+DL	DL	ATV	D+ATV	LL+DL	DL	ATV	D+ATV
350x250x37 timber	5.1E+06	8.93E+08	38.3	19	22	17	37	31	7	17	24

HANDRAILS POSTS FOR BRIDGE BOOK FCE/0535 - NYMP

Input in RED — Spacing Allowed

POSTS

	A		B		C	D	E	
					INPUT	OUTPUT		
Loading Conditions	load	load	leverarm	leverarm	fixing	Moment	height	
Pedestrian Normal	0.74		1.1			**0.81**	1	Most rural BD52/93 para 2.19
Pedestrian Crowd	1.4		1.1			**1.54**	1	Car parks and campsites
Livestock	1.3		1.25			**1.63**	1.25	BD 29/87 para 4.2 CC for 1.25
Cycles & led Horse	1.4		1.15			**1.61**	1.4	BD29/87- 4.1.4 & BE5 for 1.15m
Horse & Rider	1.3	0.74	1.25	1.6	0.1	**2.61**	1.6	Non Bridleway/BHS
CHOSEN APPLIED BENDING MOMENT					**2.61**			

Material Bend Strength - permissible stresses x modulou

Material		Spacing
Softwood C16 75x75mm	0.51	0.194
Softwood C24 75x75mm	0.72	0.274
Hardwood D40 75x75mm	1.19	0.457
Softwood C16 75x100mm	0.90	0.344
Softwood C24 75x100mm	1.27	0.487
Hardwood D40 75x100mm	2.12	0.812
Softwood C16 75x125mm	1.40	0.538
Softwood C24 75x125mm	1.99	0.761
Hardwood D40 75x125mm	3.31	1.268
Softwood C16 75x150mm	2.02	0.774
Softwood C24 75x150mm	2.86	1.096
Hardwood D40 75x150mm	4.77	1.827

Material		Spacing
Softwood C16 100x100mm	1.20	0.459
Softwood C24 100x100mm	1.70	0.649
Hardwood D40 100x100mm	2.83	1.082
Softwood C16 100x125mm	1.87	0.717
Softwood C24 100x125mm	2.65	1.015
Hardwood D40 100x125mm	4.41	1.691
Softwood C16 100x150mm	2.70	1.033
Softwood C24 100x150mm	3.81	1.461
Hardwood D40 100x150mm	6.36	2.435

Notes

A Line load kN /m applied to rail at height given in column B (Countryside Comm for livestock, BE 5 for cycles, BS 6399 for Normal and crowd)
B Ht above deck at which A is applied - not necessarily to top rail required height (Ditto)
C height above or (-ve) below the deck to the top post fixing in metres (usually a + for a fixing into a kerb)
D Bending moment from load and lever arm
E This is the height of the top rail for the type of use
Spacing of posts is the maximum spacing in metres necessary for the load and material strength
Stresses and modification factors from BS 5268. Oak=12.5, C24=7.5, C16=5.3N/mm2. K2=0.8, K3=1.5, K7=1.13 - 1.17

RAILS AND INFILL PANELS BETWEEN POSTS ***(This spreadsheet is a table - no inputs are required)***

JOB and No

Guidance fromBridge Design Memoranda BD29/87 Design Criteria for Footbridges & BE5 Design of Highway Bridge Parapets

RAILS

Loading

	F	
Longitudinal Rail	1.4	kN/m run of load (table VII BE5 page 18 for P4 pedestrian parapet)
Vertical infill panel	1	kN point load over 125mm (Do)

Materials Bend Strength - Permissible stress x modulous

Member	Type		Max Span m	Member	Type		
Softwood C16 75x50	Horiz rail	0.23	***1.15***	Softwood C16 30x100	Top rail	0.37	***1.46***
Softwood C24 75x50	Horiz rail	0.33	***1.37***	Softwood C24 30x100	Top rail	0.53	***1.73***
Hardwood D50 75x50	Horiz rail	0.70	***2.00***	Hardwood D50 30x10(	Top rail	1.12	***2.53***
Softwood C16 100x50	Horiz rail	0.31	***1.33***	Softwood C16 30x150	Top rail	0.84	***2.19***
Softwood C24 100x50	Horiz rail	0.44	***1.58***	Softwood C24 30x150	Top rail	1.18	***2.60***
Hardwood D50 100x50	Horiz rail	0.94	***2.31***	Hardwood D50 30x15(	Top rail	2.53	***3.80***
Softwood C16 150x50	Horiz rail	0.47	***1.63***				
Softwood C24 150x50	Horiz rail	0.66 kNm	***1.94***				
Hardwood D50 150x50	Horiz rail	1.40	***2.83***	Softwood C24 50x50	Vert Infill	0.22	***1.00***
				Softwood C24 40x40	Vert Infill	0.11	***0.57***
				Softwood C16 50x50	Vert Infill	0.16	***0.75***
				Softwood C16 40x40	Vert Infill	0.08	***0.44***
				Hardwood D50 50x50	Vert Infill	0.47	***2.00*** **MAX 1.3m**
				Hardwood D50 40x40	Vert Infill	0.24	***1.08***
				Hardwood D50 35x35	Vert Infill	0.16	***0.77***

MAXIMUM RAIL SPAN 2.00m TO LIMIT DEFLECTION

Calculation uses 0.8 for class 3 exposure and 1.5 for short term loading

Forestry Commission Policy - where it differs with guidance

For bridges upto 5m above the river bed, make horse handrails 1250mm high and place a sign that riders stay mounted at their own risk
For bridges upto 5m above the river bed, make handrails for cycle bridges1250mm high and place a sign that riders stay mounted at their own risk
For bridges on dangerous sites or bridleways or over busy public roads make handrails 1800mm for horses and 1400mm for cyclists.
In very rural locations assume that users are "countrywise" and will not climb handrails or fall through a barrier
The only official guidance for parapets is for bridges over highways - parapets for bridges over rivers are not so critical in many ways

DECK BOARDS FOR FOOTBRIDGES IN THE COUNTRYSIDE

	PEDESTRIAN						LIVESTOCK & HORSES					
	C16		C24		D50		C16		C24		D50	
SIZE dxb	Span	Cant	Span	Cant	Span	Cant	Span	Cant	Span	Cant	Span	Cant
36x100					750	200						
125					925	250						
150					1100	300						
50x100	500	150	700	200	1400	350						
125	625	175	850	230	1500	450						
150	725	200	1000	270	1575	525	300	100	360	130	570	185
75x100	1050	300	1450	400	2050	890						
125	1300	350	1800	475	2225	960	420	150	520	175	900	270
150	1550	400	2000	550	2350	1000	470	160	590	190	1060	300
200	1900	525	2200	725	2600	1125	560	185	730	225	1350	380
100x150	2275	700	2675	950			700	220	960	270		
200	2500	900	2950	1275			875	260	1160	330		

MAX POST SPACING FOR FOOTBRIDGES IN THE COUNTRYSIDE

POST SIZE bxd	LOADING	C16	C24	D50
75X100	Normal	1100	1550	2000
	Crowd	525	750	1600
	Cycles & Livestock	500	700	1500
75x125	Normal	1725	2000	2000
	Crowd	825	1175	Do
	Cycles & Livestock	725	1125	Do
75x150	Normal	2000	2000	Do
	Crowd	1200	1700	Do
	Cycles & Livestock	1125	1625	Do
100x100	Normal	1475	2000	Do
	Crowd	700	1000	Do
	Cycles & Livestock	675	950	Do
100x125	Normal	2000	2000	Do

	Crowd	1100	1550	Do
	Cycles & Livestock	1000	1500	Do
100x150	Normal	2000	2000	Do
	Crowd	1600	2000	Do
	Cycles & Livestock	1525	2000	Do

Heights
1.1m Normal & Crowd Loading & deck heights up to 3m
1.25 Livestock & Countryside Cycles & Deck heights up to 5m
1.4m Cycles over roads & Deck heights over 5m

MAXIMUM SPANS FOR RAILS AND VERTICAL INFILL

RAIL	SIZE	C16	C24	D50
Horiz Rail	75x50	1.15	1.37	2.00
	100x50	1.33	1.58	2.31
	150x50	1.63	1.94	2.83
Top Rail	30x100	1.46	1.75	2.53
	30x150	2.19	2.60	3.80
Vert. Infill	35x35			0.77
	40x40	0.44	0.57	1.08
	50x50	0.75	1.00	2.00

C24 SAWN TIMBER BEAMS

	Pedestrian Normal 2 Beam		Pedestrian Normal 3 Beam		Pedestrian Crowd 3 Beam		Horses 3 & 5 Beam		ATV 5 Beam
SPAN	**1m deck 0.7m crs**	**1.2m deck 0.9m crs**	**1.2m deck 0.45m crs**	**1.5m deck 0.7m crs**	**1.2m deck 0.45m crs**	**1.5m deck 0.7m crs**	**1.8m deck 0.8m crs**	**1.8m deck 0.45m crs**	**1.8m deck 0.45m crs**
3	150x75	250x100	150x75	200x75	200x100	200x75	250x100	200x75	250x150
4	200x100	250x100	200x100	200x100	200x100	250x100	250x150	250x100	250x150
5	250x150	250x150	250x150	250x150	250x150	250x150	300x225	250x150	300x225
6	250x150	300x225	250x150	300x225	250x150	300x225	300x225	300x225	300x225
7	250x150	300x225	250x150	300x225	250x150	300x225	350x250	300x225	300x225
8	300x225	350x250	300x225	300x225	300x225	300x225	Unavailable	300x225	350x250
9	300x225	350x250	300x225	350x250	300x225	350x250	Unavailable	350x250	350x250

APPENDIX 10

DIMENSIONS FOR SLT ARCHES

Radii and offsets for holes etc — Inputs in RED — Outputs in GREEN

SPAN =	20000 mm					
RISE =	1666 mm					
RADIUS =	30845 mm					
SUBTENDING ANGLE =		0.660335 radians	37.8 degrees	(this is always the same for a span/rise = 12)		
LENGTH OF ARCH ARC =		20368.04 mm				
No of PLANKS	10					
LENGTH OF PLANK ARC =		2037	Subtending Angle in rads =	0.066034	PLANK LENGTH cl =	2036
No OF HOLES	3					
Centreline of holes - arc distance =		679	Subtending Angle in rads =	0.022011	HOLE CENTRES =	679
Centreline of end holes - arc distance =		339	Vertical distance betw holes =	7 mm	divide evenly above & below centreline	
No OF HOLES	4					
Centreline of holes - arc distance =		509	Subtending Angle in rads =	0.016508	HOLE CENTRES =	509
Centreline of end holes - arc distance =		255	Vertical distance betw holes =	8 mm	divide evenly above & below centreline	
End Angle Cut =		1.89 degrees				
Thickness of arch Deck =		200 mm				
Angle End Cut =		7 mm				
Longest Plank Length for Schedule =		2043				

ARCH SHAPE	0.1	0.125	0.166	0.2	0.25	0.3	0.33	0.375	0.4	0.5
Span increments	2000	2500	3320	4000	5000	6000	6660	7500	8000	10000
Rise at span points	600	729	923	1066	1250	1399	1480	1562	1599	1666

NOTES

Arc length = angle in radians * radius

radians/0.01745 = angle in degrees

Chord length -= 2*radius*sin(subtending angle/2)