University of Southern Queensland

Faculty of Engineering and Surveying

The Maintenance of "Moment Continuity" within Structural Concrete Beam Steps.

A dissertation submitted by

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In fulfillment of the requirements of

Courses ENG4111 and 4112 Research Project

Towards the degree of

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FOREWORD

In most structures the continuity or strength between two adjacent members is necessary even though the members may meet perpendicularly or axially offset. The internal forces generated at such a ‘step’ may cause failure within the joint before the design strength of the beam itself is attained. The step efficiency is therefore critical to the overall beam design.

Previous research has shown that foundation steps often have serious strength deficiencies. It is the intent of this project to research the efficiency of ‘incorrect’ and ‘correct’ step detailing.

The provision for correct detailing of steps in reference to Australian Standards has been found to be limited in many criteria, especially overall height. It is the objective of this project to analyse these code limitations and compare such with mathematical and physical test results.

To gain a greater understanding of the effects that steel reinforcing has within the ‘step’, this project will analysis, construct and destruct to failure a series of stepped concrete beams with differing reinforcing arrangements. Use of this test data will promote more accurate design of concrete beam steps in order to maintain moment continuity.

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______________________ **(28 / 10 / 08)**

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NOMENCLATURE AND ACRONYMS

The flowing abbreviations have been used throughout the text and bibliography:-

D		Overall depth of foundation.
AS1012.9	=	Australian Standard for Methods of testing concrete.
AS2870	=	Australian Standard for Residential slabs and footings.
AS3600	=	Australian Standard for Concrete structures.
a	=	depth of concrete stress block.
A_{sc}	=	area of steel in compression.
A_{st}	=	area of steel in tension.
$A_{sv,min}$	=	minimum area of shear steel.
B	=	width of section.
b_v	=	effective width of section for shear.
C	=	compression force.
C_c	=	compressive force of concrete.
C_s	=	compressive force of steel.
d	=	depth of section.
d_{st}	=	depth of steel in tension.
d_{sc}	=	depth of steel in compression.
d_n	=	depth to the section neutral axis.
d_o	=	Distance from extreme compression fibre of concrete to centroid of tensile steel.
E	=	Elastic Modulus.
f'_c	=	Compressive strength of concrete.
f'_{cf}	=	Flexural tensile strength of concrete.
f_n	=	Function of.
f_r	=	Applied force to rupture.
f'_{sy}	=	Yield strength of reinforcing steel.
f'_{su}	=	Ultimate strength of reinforcing steel.
f_u	=	Maximum applied force.
f_y	=	Force to yield.
I_{cr}	=	second moment of area of the cracked section.
I_{ef}	=	effective second moment of area.
I_g	=	second moment of area of the gross concrete cross-section.
kN	=	Kilo Newton
L	=	length

$L_{sy.t}$	=	minimum development length to achieve steel yield stress.
M	=	moment.
M*	=	design moment.
M_{cr}	=	cracking moment.
MPa	=	Mega Pascal.
M_u	=	ultimate moment.
M_y	=	yield moment.
N*	=	design axial force.
N20	=	design concrete strength (example 20 MPa)
n	=	modular ratio, E_s/E_c
p	=	percentage of steel in section.
Pu	=	ultimate applied load.
Py	=	yield applied load.
V	=	shear force.
V*	=	design shear force.
Vuc	=	ultimate shear strength.
Vu,min	=	ultimate shear strength provide with minimum shear reinforcement.
s	=	spacing of shear reinforcing steel.
T	=	tension force.
B_1,β_2,β_3	=	multiplying factors (Chapter 3)
Δ	=	deflection.
Δ_r	=	deflection at rupture.
Δ_u	=	deflection at ultimate.
Δ_y	=	deflection at yield.
ε_{sc}	=	compressive strain in compressive reinforcement.
ε_{st}	=	tensile strain in tensile reinforcement.
$\sigma_{st.y}$	=	stress of tensile reinforcing steel at yield.
$\sigma_{st.u}$	=	stress of tensile reinforcing steel at ultimate.
ø	=	strength reduction factor.

CHAPTER 1

INTRODUCTION TO MOMENT CONTINUITY

“A chain is only as strong as its weakest link.”

(Pauly, 1975)

1.1 Outline of the study

The above statement although referring to a chain, can be directly applied to the physics of Structural Engineering, whereby a structure will be only as strong as its weakest point. A step in a structural concrete beam has been identified as a potential ‘weak link’, which suggests the need for greater understanding behind its mechanics.

The purpose and scope of this study is detailed in 1.4 Research Objectives.

1.2 Introduction

The *“The Maintenance of Moment Continuity within Structural Concrete Beam Steps”* is an ‘own project’ topic chosen from exposure to real world situations that have demonstrated deficiencies in both existing and current structural foundation designs.

This study aims to achieve a much clearer understanding of the effects that steel reinforcing arrangements have to the final efficiency of concrete beam steps, with reference to adjoining member’s ultimate bending moment. By constructing and loading scaled concrete beam steps, final data shall measure and compare crack, yield and ultimate failure against that of different arrangements of reinforcing steel within the step.

1.3 The Problem

This project topic was chosen due to the professional exposure of many 'failed' or poorly designed concrete foundation steps observed on a regular basis from onsite and professional engineering situations. An example of the effect a failed foundation step may develop can be seen in Figure 1.1. Such failure can and most likely will result in unsightly building damage and it is for this reason that it is proposed to research the methodology, theory and performance of correct and incorrect step detailing in structural concrete beam locations.

Figure 1.1 *Failed foundation step.*

Previous research over many years has shown concrete beam steps have serious strength deficiencies. But how great are these deficiencies, and what is needed to rectify such points of poor strength? It is the intent of this study to research the efficiency of 'incorrect' and 'correct' step detailing compared to that of a singly supported, straight test specimen of similar cross-sectional size, span and percentage of steel reinforcing. Using this data, it will compare design clauses in both AS3600, AS2870 (Concrete Structures and Residential Slab and Footings Australian Standards respectively) and proficient engineering principles to potentially expose areas of poor knowledge and design.

1.3.1 Penalties and provisions in AS3600/AS2870 for Concrete Beam steps.

1.3.1.1 AS2870

The provision for correct detailing of structural concrete steps in reference to Australian Standards is limited to say the least. Figure 1.2 is an extract from AS2870 - The *Residential Slab and Footing Code*, which outlines the suggested methods for foundation steps.

This extract is the only standardised literature for concrete steps when used in a foundation design situation. These arrangements will be analysed and compared with mathematical and physical results from testing data.

The base of a strip footing shall be horizontal or at a slope of not more than 1 in 10, and the footing shall be stepped in accordance with one of the methods given in Figure 5.6.

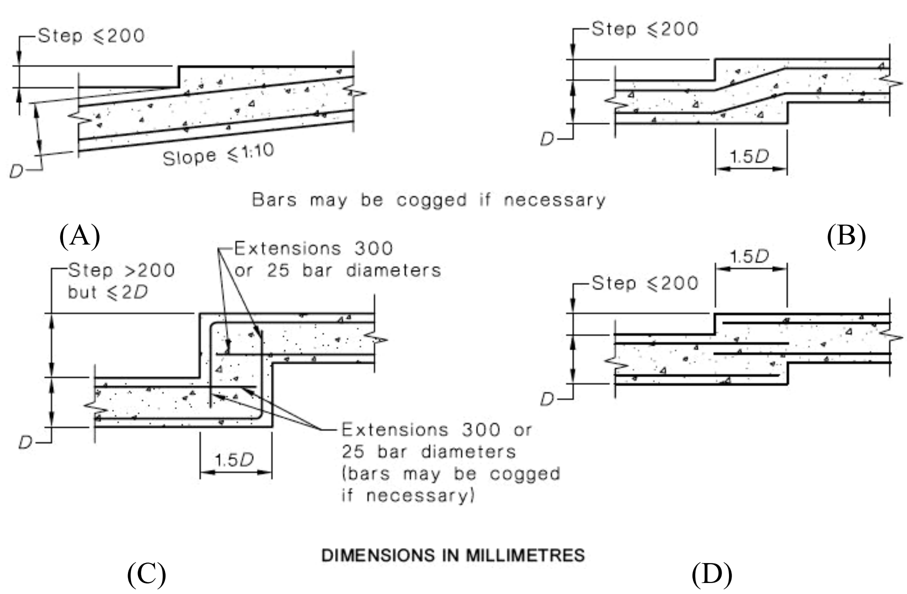


Figure 1.2 *AS2870 suggested step arrangements.*

Such results will aim to prove or disprove the current Australian Standard literature in respect to overlap, step height limits and the size and arrangement of reinforcing steel. Pending test results, such step limits as defined by AS2870 may be potentially increased to accommodate larger foundation steps as commonly required in many real life situations.

1.3.1.2 AS3600

Specifications defined in AS3600 can be limited to static analysis of reinforced concrete beams using '500N' bar reinforcing steels (500 MPa yield stress), minimum percentage of steel (*p*), and concrete cover to reinforcing and other clauses to follow.

1.3.2 Past Research and Results

Study on the arrangement of reinforcing steel for knee joints has been previously conducted with varying results. Evidence of step or corner details have been found dating back to 1969 and potentially beyond. Such test data (Warner, R.F et al, 1999) has outlined corner efficiency of as low as 10-30% in various reinforcing arrangements. This efficiency value is based on the joints ability to allow adjacent members to develop their full strength. Such connections and their associated efficiency results obtained by Warner can be observed in Fig 1.3. Further testing a decade later has shown that 100% efficiency can be obtained by the use of complex steel details, as shown in Fig 1.4 (Warner, R.F. et al).

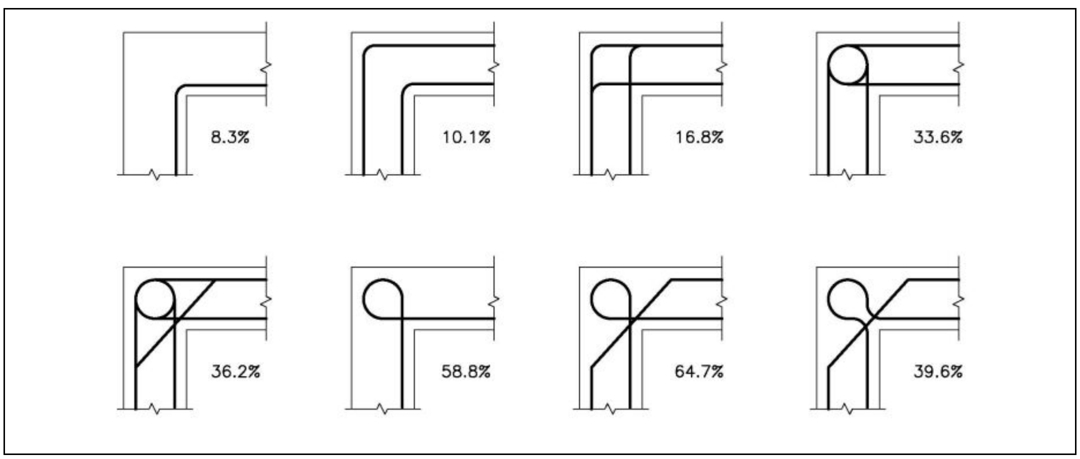


Figure 1.3 *Previous efficiency results.*

Although many of these previous studies have accomplished detailing of knee joints (closing and opening) to achieve 100% member efficiency, these details are both complex, time consuming and difficult to construct in a typical residential or smaller scale commercial foundations. The importance and consequential effects of incorrect foundation step mechanisms for multistory high-rise or low budget residential situations could be considered of similar importance.

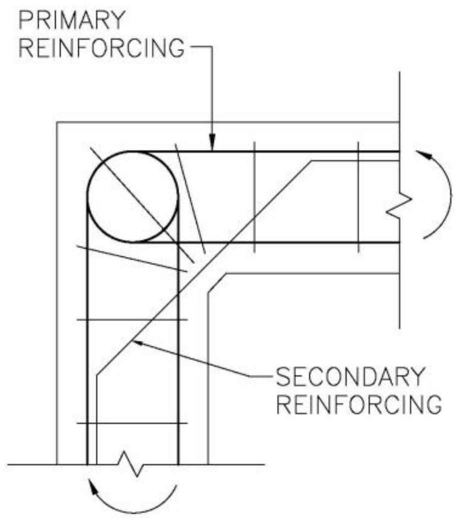


Figure 1.4 *Previous 100% efficient knee joint detailing.*

1.4 Research Objectives

Objective for this project are defined as the:

1. Research of the fundamental engineering principles on structural concrete beams, and the 'moment' forces produced on critical joint locations. (Refer Chapter 2).

2. Design scaled reinforced concrete beams of various step configurations used in common real world structural examples. (Refer Chapter 3)

3. Construct and load to failure scale reinforced concrete beams, detailing all load, stress-strain and ultimate load/deflection characteristics to various step configurations. (Refer Chapter 4)

4. Analyse and compare results of such testing with preliminary design calculations. (Refer Chapter 5)

5. Evaluate and define optimum steel reinforcement configurations to maintain maximum moment continuity at structural concrete beam steps. (Refer Chapter 5)

6. Prove or disprove current AS 2870 step limitations/configurations. (Refer Chapter 5)

1.5 Conclusions: Chapter 1

This dissertation aims to develop increased understanding of structural concrete steps by constructing and testing 5 reinforced concrete beams of varying reinforcing steel arrangements to identify and compare the strength efficiencies of each.

The research is expected to produce test results that will define how, where and why such a mechanism may fail. By analysis of results, numerical computations backed by practical performance will aim to detail methods of improving the design and detailing of a concrete beam step in financial, constructional and economical efficiencies.

A review of literature for this research will identify the basic principles behind properties of concrete, the physics of structural steps and explore the reasons as to why such concrete steps in building foundations may fail.

The results of this study will then be used to amend if necessary the foundation step details currently in use by structural engineering firms such as Reid Consulting Engineers Pty Ltd and to distribute the summarised results to varying industry professionals including QBSA, Builders, Concreter's, Council authorities and the like.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

To understand the reasons for undertaking research on moment continuity in concrete beam steps, it is necessary to outline the basic principles behind the term 'moment continuity', structural steps, reinforced concrete behaviour, and the forces exerted on foundations to develop deficiencies in a stepped beam arrangement.

Following this chapter, the methodology of testing configurations will be defined and discussed.

2.2 Moment Continuity

Reference of the term 'moment continuity' in the title of this research project relates to the members ability to transfer moment resistance through an entire member, regardless of its shape.

The concept of moment (or bending moment) in engineering is an internal force trying to pull open part (or all) of a member dependant on how the member is supported and/or loaded. This force is developed by the introduction of a bending action placed on the member by dead, live, wind or soil pressure loading. Figure 2.1 demonstrates the basic principles behind bending moment.

This concept of bending moment induces tensile and compressive stresses in the member, which as will be covered later will influence where and how much reinforcing must be applied to the member. Failure in bending will occur when the bending moment is sufficient to induce tensile stresses greater than the yield stress of the composite material. It is possible that failure of a structural element in shear may occur before failure in bending; however the mechanics of failure in shear and in bending are different.

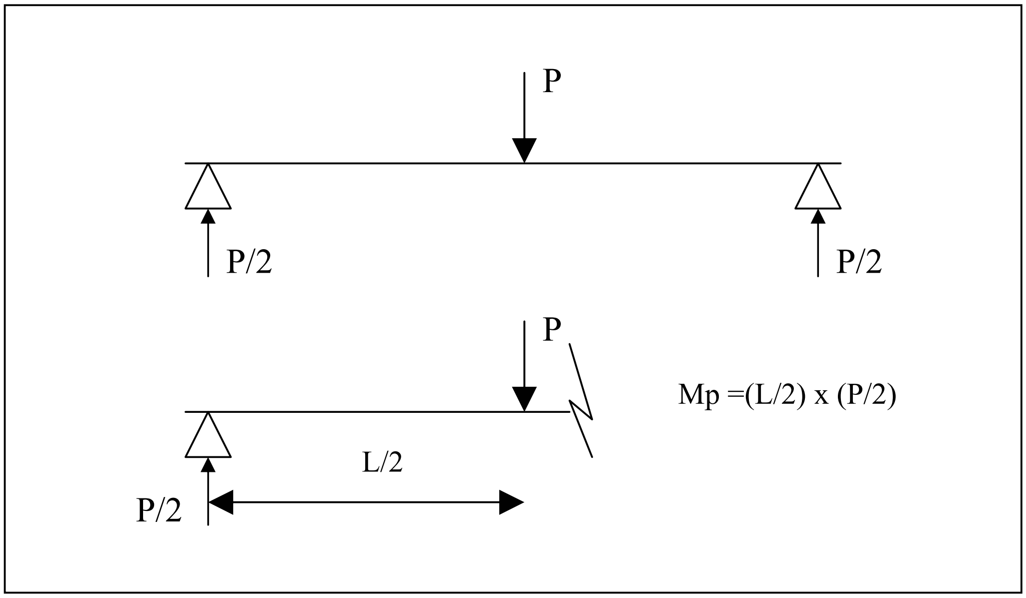


Figure 2.1 *Basic principles of Bending Moment.*

The ability of a concrete member to resist bending moment or maintain moment continuity will be investigated in Section 2.3.

2.3 Reinforced Concrete Properties

The study of steps in structural concrete beams is commissioned by the frequent use of reinforced concrete for many uses in today's construction industry. From multi-storey buildings to a backyard shed, concrete is used for its availability and price compared with other materials. It does however lack tensile strength, which can be overcome by the use of steel reinforcement. This is generally in the form of deformed steel bars varying in size from 6 mm up to above 40mm depending on the resistance forces required. This concrete and steel bonds together to form a composite structure, with the ability to maintain large loads if detailed correctly.

In practice, a structural concrete member will under go differential forces throughout the length of beam. It is these forces of compression, tension, shear and twisting that require the placement of steel reinforcing to differ at various segments of the member. In compression the concrete is efficient, removing the need for steel reinforcing for strength in most circumstances. In tension and shear however, the concrete's strength is virtually redundant. The strength of the beam is determinant on the size and location of the steel reinforcing in tension and shear areas.

In the scaled model testing to follow, analysis of the structural step will be carried out using the mechanics of knee joints as previously discussed. The arrangement of the abovementioned tensile steel will play a fundamental role in the overall efficiency of the step in relation to the adjoining member strength.

In structural foundations, the use of reinforced concrete takes the job of resisting forces in multiple directions. It is for this reason that the design of reinforced concrete is these situations must include both upper and lower steel reinforcing .

In testing of the concrete members, there are typically a number of characteristic points that define the behavior of the member itself. These include the cracking moment, the yield moment and ultimate moment. The cracking moment is the load applied that will initiate concrete cracking and put the reinforcing steel into service. The yield moment occurs at the point of applied load where the behaviour of the beam and reinforcing steel change from elastic to plastic performance. From this point onwards, the beams shape will not return to its original state if the loading is removed. Ultimate moment is the maximum applied load that the member has withstood. Although this is typically larger in capacity then yield, it is a point of character that is not desired in serviceability.

2.4 Steps in Structural Foundations

A structural step can be described as a vertical or horizontal offset of a structural member in which internal forces are maintained through the 'step'. The step can be divided into two mechanisms' in which the internal forces are redistributed. These are commonly termed *knee joints* seen in Figure 2.2. Examples of step or knee joints can be found in many common structures around the neighborhood such as the base of block retaining walls and the column-beam portal frame connection. Further examples can be found at the base corners of concrete swimming pools and building platform steps. For the purpose of this study, it is assumed that a typical concrete beam step will follow a shape similar to Figure 2.2.

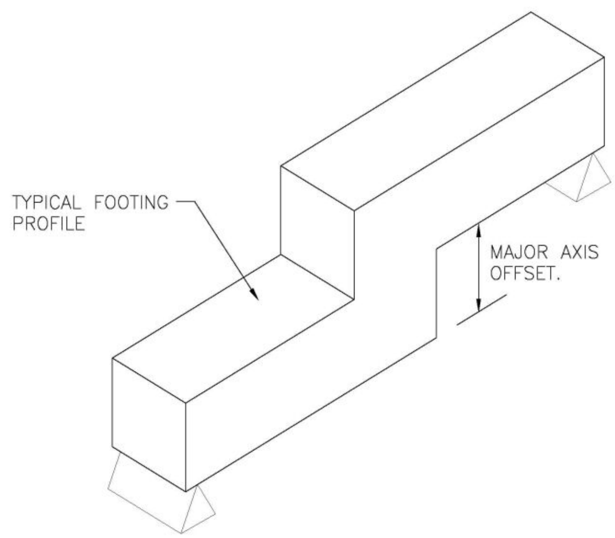


Figure 2.2 *Typical step shape.*

In a building situation, such joints typically connect a horizontal member (or part thereof) to a vertical member (or part thereof), or a connection required to change directions sharply. These connections are commonly recognized as weak spots in most designs and are often exploited during such failure modes as earthquake loading. It has been described that "previous scale testing has confirmed individual components remained relatively intact and that collapse is nearly always attributable to connection failure" (Park & Pauley, 1976). These common deficiencies suggest care is needed to achieve satisfactory step design which forms the basis of this research project.

Park and Pauley (1976) suggest that the requirements for satisfactory performance of a joint can be summed up as follows:

1. A joint should exhibit a service load performance equal in quality to that of the members it joins.
2. A joint should possess a strength that corresponds at least with the most adverse load combinations that the adjoining members could possibly sustain, several times if necessary.
3. The strength of the joint should not normally govern the strength of the structure, and its behavior should not impede the development of the full strength of the adjoining members.
4. Ease of construction and access for depositing and compacting concrete are other prominent issues of joint design.

These requirements of performance will form the basis of overall objectives in mathematical calculations and practical testing to follow.

2.4.1 Step Height Restraints

Although the designated design literature of AS2870 for the maximum step height is restricted to two times the footing depth (2D) as seen in Figure 1.2(c), the requirement for additional step height is commonly requested.

From bending moment analysis of a stepped simply supported member, it can be observed from Figure 2.3 that the applied bending moment load to the stepped member is similar to the adjacent horizontal supported members. Assuming the location and magnitude of applied load and hence bending moment remain similar, the forces will not increase with height. Design to adjoining knee joint mechanics in respect to steel reinforcing detail and development lengths/locations is designated as the critical factor in the increase or decrease of step heights.

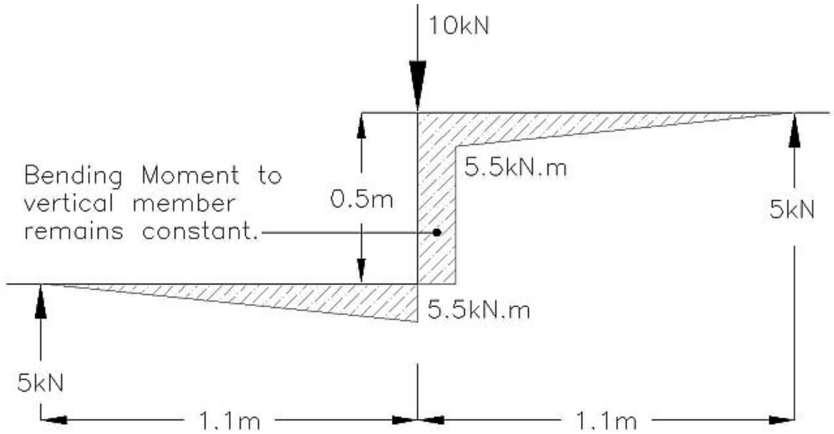
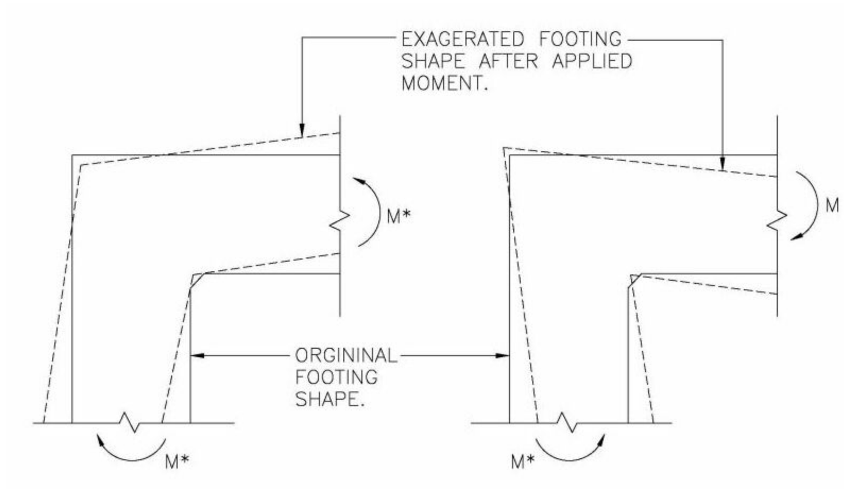


Figure 2.3 *Step bending moment diagram.*

2.4.2 Knee Joints

Previously it was stated that a typical structural step could be categorised into 2 individual mechanisms' called knee joints due to the shape and operation similar to a human knee. The mechanics of a knee joint can then be divided into a further 2 categories; open and closing knee joints. This categorization is dependant on the sense of the applied moments or simply the direction of movement. An open knee joint occurs when the bending moment is positive, whilst a closing knee joint operates when bending moment is applied in a negative manner. Figure 2.4 defines such joints respectively.



Positive 'Opening' Moment | Negative 'Closing' Moment

Figure 2.4 *Effects to knee joint from applied moments.*

2.4.3 Opening Knee Joint

As shown in Fig 2.4, the general deformation within an opening knee joint arrangement is due to positive bending moment. This moment attempts to compress external beam faces, whilst tensioning internal locations. The opening joint is recognized as the most severe due to the location of compression and tension within the joint core. These actions when simplified, produce a resultant tensile force which attempts to 'punch out' the internal corner concrete. This phenomenon, combined with poor steel detailing, can produce seriously defective joints. Fig 2.5 attempts to detail the internal forces within an open knee joint.

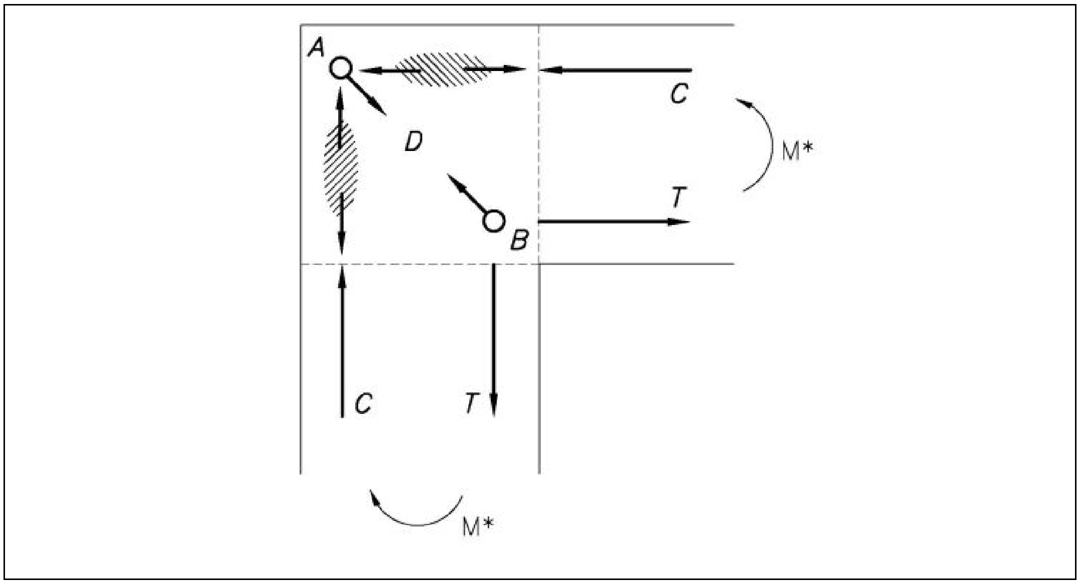


Figure 2.5 *Open knee joint internal forces.*

The compressive force C shown in Fig 2.5, cannot turn the corner at A without the addition of tensile reinforcement along D. Likewise at B, the resultant tensile force indicates the requirement of diagonal reinforcing to resist cracking and ultimate failure of the member/joint. Previous research has indicated as the secondary reinforcing steel arrangement can prove impractical in many residential or commercial structural step locations. It is for this reason that research into obtaining maximum joint efficiency with minimum detailing of steel through the member is of great importance.

2.4.4 Closing Knee Joint

The other knee joint mechanism is subjected to a "closing" action or bending moment which was outlined previously in Fig 2.4. In this arrangement, the outer bars subjected to tension are continuous with sufficient anchorage and generally can develop full strength. In order to prevent concrete crushing, the outer tensile bars should bend the corner in a radius as large as possible, whilst maintaining an adequate lever arm for moment resistance. In biaxial operation, the inner reinforcing bars operate in compression which anchorage is not critical. As per the opening knee joint, the resultant free body forces acting on the closing joint are shown in Fig 2.6. These forces introduced into the joint core are in the form of shear due to anchorage friction, and when approaching the concrete's tensile strength (approximately 3-4 MPa with N20 concrete) diagonal cracking may start to develop.

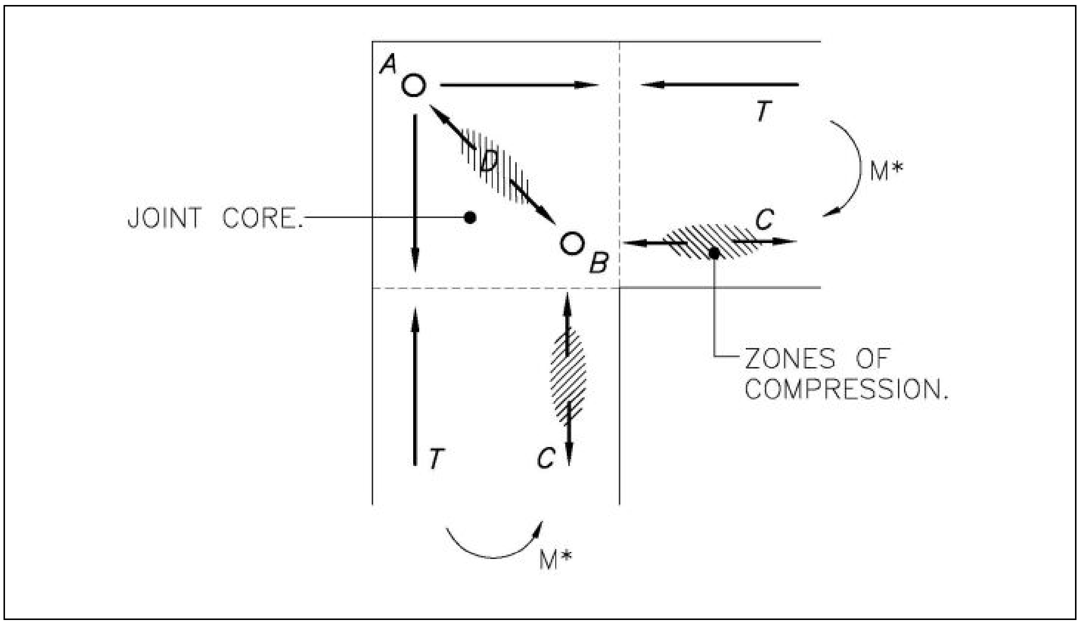


Figure 2.6 *Closing knee joint internal forces.*

In this study, due to the scaled size of test specimens, no attempt will be made to control the development of diagonal cracking by the use of secondary reinforcement.

2.5 Soil Pressure

An element of force subjected to concrete foundations can be by sub-surface soil shrinking and swelling. This is caused by changing moisture conditions within the soil. Moisture added to a dry soil tends to swell and increase the soil's unit volume, whilst drying of a saturated soil will decrease the soil's unit volume. If this process occurs at different rates (either shrinking and swelling, or varying rates of shrinking or swelling) over and around the area of a structure, then the foundation system will undergo a termed called differential movement. This generally is found in expansive clay soil types that can shrink or swell up to +150mm at natural surface level. In real world examples, the effects of differential shrinking and swelling can be initiated simply by rain or sunshine. This process, although exaggerated for visual effect, can be seen in Figure 2.7 below.

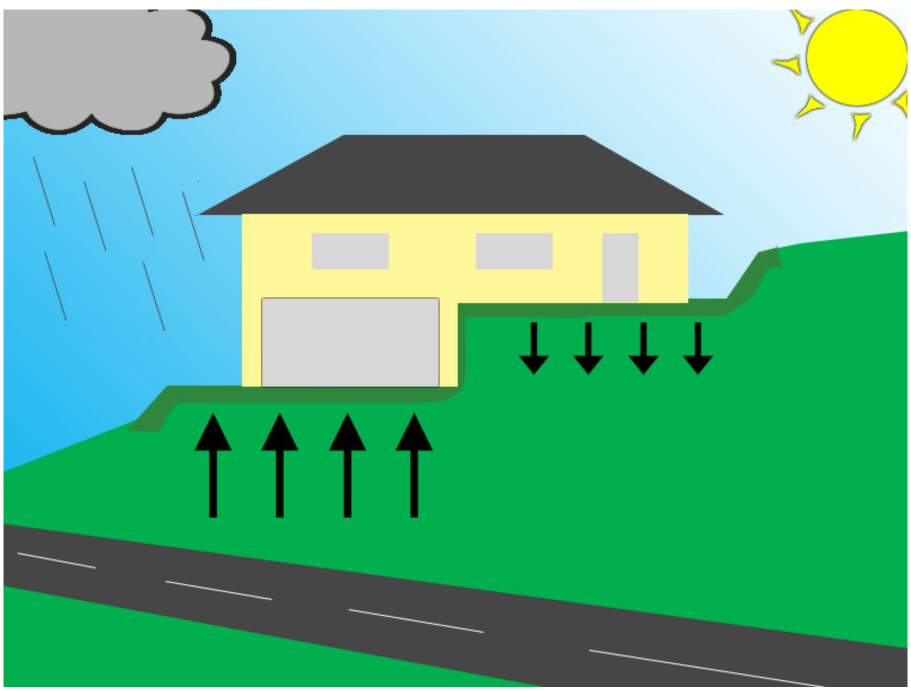


Figure 2.7 *Soil moisture related ground movements.*

The effect of soil pressure on a foundation is a constantly changing force and very difficult to model over the area of a structure, yet regardless of the direction of shrink or swelling, this movement does produce a bending moment in the foundation beams. This can be replicated by the effect of a single point load on a simply supported member and it is this theory will form the basis of this project testing.

2.6 Conclusions: Chapter 2

From this theoretical exploration it is noted and observed that serious deficiencies exist in the moment maintenance of structural concrete members containing a knee joint or step mechanism. It is critical that the understanding and practice of step detailing is effective. The above text offers an overview of the mechanics involved with such, and the testing of scale models to follow, will intuitively search for practical and efficient forms of steel detailing.

CHAPTER 3

METHODOLOGY

3.1 Test Planning

It is proposed that testing will consist of designing, constructing and loading a series of scaled concrete beams. These scaled beams will include 1 straight simply supported ‘pilot’ beam and a further 4 stepped beams of similar cross-sectional area. The 4 stepped beams will all differ in terms of steel reinforcing arrangement within the step envelope yet will retain similar overall steel to that used in the pilot beam. Dividing the 4 stepped beam arrangements will see 2 examples of what is considered to be ‘good’ detailing, and the remaining 2 beams of what could be expected as ‘bad’ details. The pilot beam will act as the member that all other beams desire to match in performance as stated by Park and Paulay in Item 1 of the performance requirements.

To arrive at this point we must employ both empirical and theoretical techniques. By understanding the properties of reinforced concrete beams, design and testing of experimental specimens will adequately provide us with the data required to analyse the performance of structural concrete steps.

The methodology for testing will define calculations in the pilot beam design, theoretical maximum loading values, step beam design including varying reinforcing arrangements and the final construction details of each (Including the construction and loading of test cylinders for concrete strength measurement- AS1012.9-1999) with efficiency comparison the final objective. All of this work will follow closely with AS3600 and AS2870 respectively. From test results, a simple insight into what works and what doesn’t can be attained. This knowledge can then be applied to professional real world situations.

3.2 Beam Design

3.2.1 Prerequisites.

3.2.1.1 Size

The design and size of the test beams will be critical for gathering results that are achievable by the testing equipment and also provide data that is easily manipulated for post testing mathematics. If the beam is to small then the results may suffer from magnified friction factors and overall working tolerances to small to overcome. To large and the procedure of testing becomes difficult in terms of construction, transport, final failure loads and also price to form and finish. For transportation (via a standard utility vehicle) the beam length must be less than 2.5 metres. For testing setup, utility loading and unloading, and general movement, the beams must have an overall mass capable of lifting by 3 adults. These factors will form the basis of final beam dimensions and reinforcing steel size and placement.

3.2.1.2 Loading

The beams are to be loaded in the form of '3 point loading' at mid-span on the member, central to the vertical 'leg' of the step as per Figure 3.1. As previously discussed in Section 2.5, this loading setup is not an accurate model of soil forces generated, but is considered a sufficient mechanism to test the performance of varying step reinforcing arrangements in flexural performance.

The point load is to be delivered via a measured hydraulic ram which will log deflection vs. load components. The specifications of the testing equipment allow ultimate or failure loads of the test specimen to be in excess of 450kN, although it will be designed for much lower values.

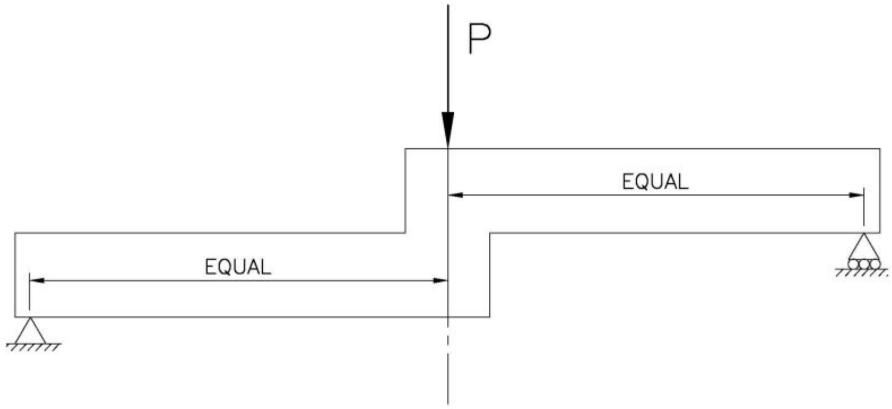


Figure 3.1 *Location of loading.*

3.2.1.3 Reinforcement steel properties.

To gain a more accurate prediction in the beams flexure and deflection, the testing will use the actual yield stress of the reinforcement. This will be performed by the use of tensile testing of the 500N bars. A yield stress of above 550 MPa is expected for the steels actual f'_{sy}, and an ultimate stress in excess of 625 MPa. Figure 3.2 shows typical stress/strain characteristics of the 500N bars.

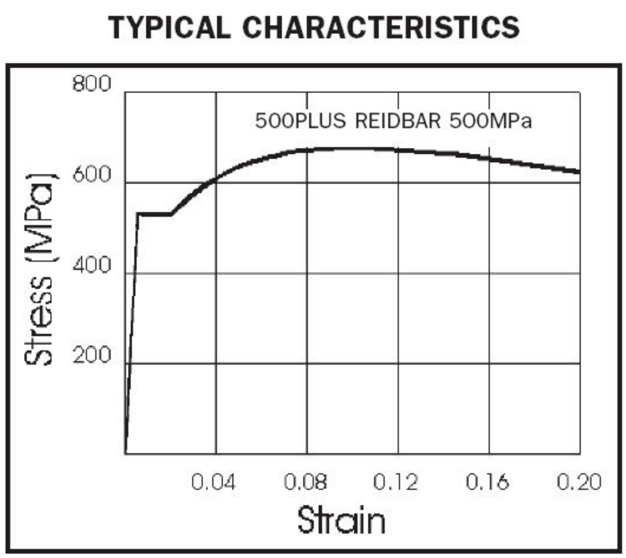


Figure 3.2 *500N performance stress.*

3.2.1.4 Tensile steel ratio.

AS3600 Clause 8.1.4.1 places a lower limit on the steel content in a structural beam section. The requirement is that M_u be at least 20 per cent greater than the cracking moment M_{cr}. This is to avoid facture of the steel and sudden collapse on cracking. AS 3600 defines the limit to satisfy the above requirement as:

$$p = \frac{Ast}{b \times d} > \frac{1.4}{Fsy}$$

Previous tests to the research of knee joint efficiencies have indicated differing results based on the percentage of steel in the beam section. Tests from the *University of Nottingham* (2001) have indicated that full moment capacity was attainted by percentages of steel (p) as little as 0.75%. On the other hand, joints studied by *Park (1976)* with p = 3.0% failed at a load less than 80% of the adjoining members flexural capacity. It is shown that the tensile steel ratio is dependant on the arrangement of such reinforcing within the joint itself.

For the purpose of this project, compliance of the tensile steel ratio for preliminary design shall be to AS3600 requirements and formulas.

3.2.1.5 Concrete Strength

For this testing, concrete compressive strength of 20MPa will be used. This reflects similar footing concrete strength used in most in situ residential and commercial developments. Local suppliers will deliver a single batched concrete order for all 5 beams to ensure consistency. Compressive testing cylinders are also to be poured for final compressive strength test.

3.2.1.6 Ease of Construction

The final condition of design relates mostly to the stepped beam. Ease of construction of any steel detailing must be achievable by on site Builders, Concreter’s or Laborer’s. This means all steel work is simplified to ensure replication of designs by workers can be easily understood, and placed with acceptable tolerances. Excessive detailing of the step envelope although most likely giving a greater efficiency will result in increased preparation time, and an increased probability of incorrect placement.

3.2.2 Pilot Beam Design

3.2.2.1 Prerequisites of Pilot Beam Design:

- Proposed failure loads (fn[Length,Ast])→ *must not be to large or small.*
- Overall Mass (fn[Length]) → *for logistical reasons.*
- Tensile Steel ratio in section → *equivalent to full scale footing.*

Putting this into practice:

- Trialing a 1/3 scale factor of typical 300x600 residential footing. (2/N16 Tensile Reo) → 100 x 200 section size.

3.2.2.2 Steel Ratio

The requirements for tensile steel selection are:

- 500 MPa minimum yield stress.
- > 45mm² cross-sectional area.
- 'Ribbed' series reinforcing bar for anchorage/friction requirements.
  AS3600 Percentage of Steel in section (p) = Ast / bd

$$P\left(full\,scale\right) = \frac{402mm^2}{300x600} = 2.3\,x\,10^{-3}$$

$$A_{st} = \left(2.3 \times 10^{-3}\right) \times (100 \times 200) = \mathbf{45}\boldsymbol{mm^2}\ \textit{or}\ \mathbf{7.5}\boldsymbol{mm\,Ø}$$

Due to difficulty of supply, Ribbed Wire **RW9.5** (9.5mm Ø – 71mm2) was adopted. This 'bar' still retains 500MPa yield stress, and it was felt a slightly larger percentage of steel would return more realistic loading performance.

3.2.2.3 Theoretical Moment Capacity

The following process defines the nominal moment capacity at failure for the doubly reinforced section.

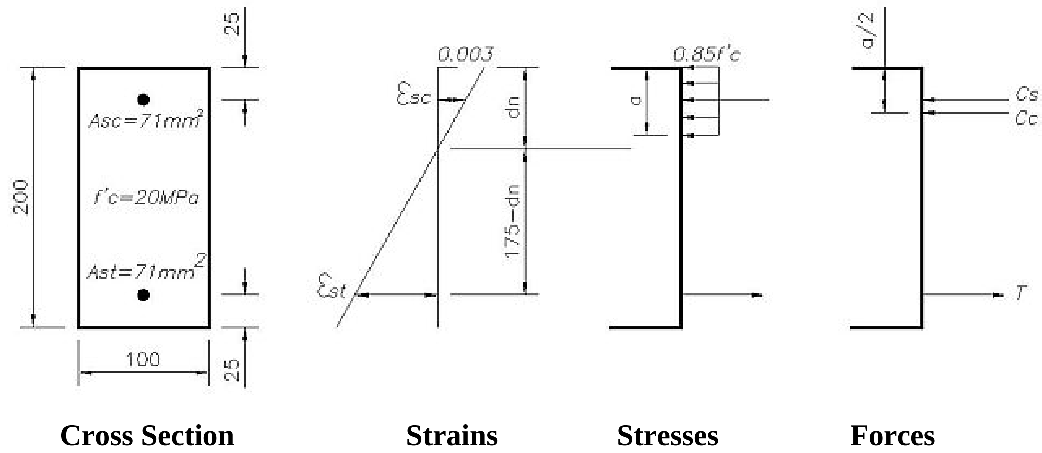


Figure 3.3 *Section properties.*

$$\textit{Yield Moment Capacity } My = \textit{Sum of forces about top concrete fibre}$$

$$= (T \times 175) - \left(Cc \times \left(\frac{a}{2}\right)\right) - (Cs \times 25)$$

The assumption is that the compressive steel is not at yield. The condition of Cc+Cs–T=0 will result in a quadratic eqn. Instead trial and error was used to solve for dn. (Depth to neutral axis).

Using dn = 23.35 (from trial and error);

Compressive Steel Strain: $\varepsilon_{st} = 0.003\left(\dfrac{23.35 - 25}{23.35}\right) = -0.00212$

Compressive Steel Force: $C_s = -0.00212 \times 200000 \times 71 = -\mathbf{3}\boldsymbol{kN}$

Concrete Compressive Force: $C_c = \left(0.85 \times f'c \times b \times \gamma\right) \times dn$

$= 1.65 \times 23.35 = \mathbf{38.53}\,\boldsymbol{kN}$

Tensile Steel Force: $T = 71 \times 500 \quad = \mathbf{35.5}\,\boldsymbol{kN}$

Checking Equilibrium:

$$\Sigma H = 38.54 - 3 - 35 = \mathbf{0.53}\,\boldsymbol{kN}$$

This indicates that the 'Compressive Steel' is actually operating in <u>Tension</u>! Calculating back to find depth of concrete stress block 'a' we find:

$$1.94a = 1.64d_n$$

$$\therefore a = \frac{1.64 \times 23.25}{1.94} = 19.86\,mm$$

Calculate M_u, taking moments about the top fibre:

$$M_y = (35.5 \times 175) - \left(38.53 \times \left(\frac{19.86}{2}\right)\right) + (3 \times 25) = \mathbf{5.9\,kNm}$$

Checking steel in tension yields before concrete crushes:

$$\varepsilon_s = 0.003\,\frac{175 - 23.35}{23.35} = 0.019 >> 0.002$$

Therefore it is concluded that steel will yield far before concrete crushes and the section is considered ductile or under-reinforced. Following section design, the span can now be determined as follows.

From similar methodology, adopting an ultimate stress of 650 MPa for the 500MPa yield stress rated reinforcing steel:

Dn	= 26.53 mm	(Compressive steel now in compression!)
a	= 22.5 mm	
f'st	= 650 MPa	
f'sc	= 34.8 MPa	(Compressive stress in steel)
f'c	= 46.1 MPa	(Compressive strength in concrete)

M_u = ***7.52 kN.m***

This figure although not the moment causing rupture, should represent the applied maximum moment capacity the section can withstand.

3.2.2.4 Load width

- Must be transportable by a standard utility vehicle → L < 2.5m
- Final 3 point load capacity a function of length, steel and section size.

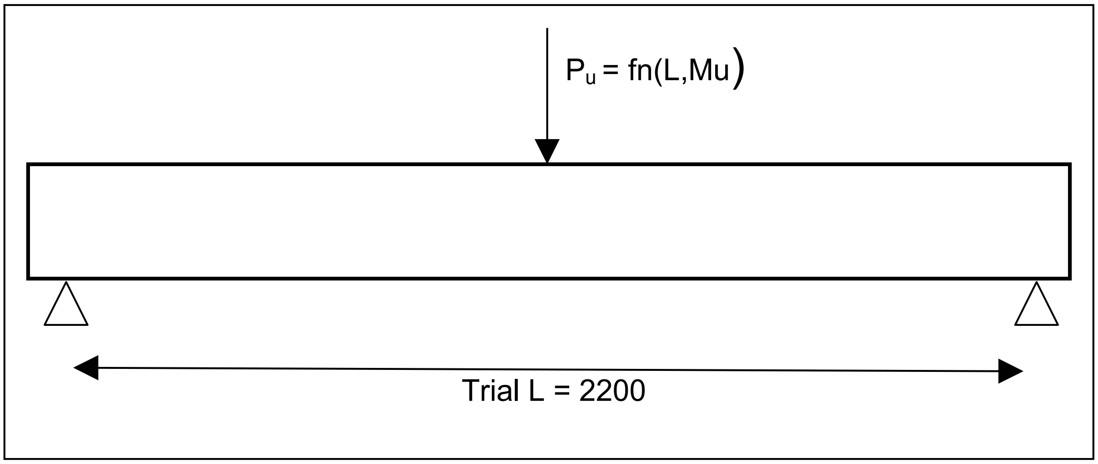


Figure 3.4 *Section span.*

$$Trialing\ L = 2200,\ Py = \frac{My\ x\ 4}{2.2} = \frac{5.9\ x\ 4}{2.2} = \mathbf{10.73\ kN}$$

$$Trialing\ L = 2200,\ Pu = \frac{Mu\ x\ 4}{2.2} = \frac{7.52\ x\ 4}{2.2} = \mathbf{13.67\ kN}$$

This calculated point load to yield and ultimate failure is considered acceptable for testing conditions.

A progressive summary of the Pilot beam specifications are defined as:

- 100 x 200 section size.
- 20 bottom cover to steel reinforcing.
- 9.5 Ø Ribbed Wire reinforcing steel.
- 2200 supported span width.
- N20 concrete.

3.2.2.5 Deflection

In construction of foundations, the effects of significant deflection may result in excessive and unsightly building damage. Theoretical deflection calculations of the pilot beam expected at ultimate load are as follows:

Deflection at failure load F_u: $\Delta_u = \frac{1}{48} \times \left(\frac{F_u \times L^3}{EI_{ef}} \right)$

Where, $I_{ef} = I_{cr} + \left(I_g - I_{cr}\right)\left(\frac{M_{cr}}{M_u}\right)^3$ and Fu= 13.67 kN; Mu =7.52kN.m

And $I_{cr}, I_g \; and \; M_{cr} = ?$

Calculating cracking moment M_{cr};

$$M_{cr} = f'_{cf} \times \frac{I_g}{y_t}$$

And

$$f'_{cf} = 0.6\sqrt{f'_c} = 0.6\sqrt{20} = 2.68$$

$$I_g = \frac{bD^3}{12} = 100 \times \frac{200^3}{12} = 66.6 \times 10^6 \, mm^4$$

$$y_t = 100$$

Therefore

$$M_{cr} = 2.68 \times \frac{66.6}{100} = \mathbf{1.8 \, kNm}$$

Calculating cracked section moment of inertia:

$$I_{cr} = \frac{1}{3} bd_n^3 + (n-1) A_{sc}\left(d_n - d_{sc}\right)^2 + nA_{sc}\left(d_{st} - d_n\right)^2 \; where \; n = \frac{\varepsilon_s}{\varepsilon_c} = 8.5$$

$$= \frac{1}{3}\left(100 \times 23.35^3\right) + (8.5-1)\,71\,(23.35-25)^2 + 8.5 \times 71\,(175-23.35)^2$$

$$= \left(0.43 \times 10^6\right) + \left(1.5 \times 10^3\right) + \left(13.9 \times 10^6\right)$$

$$= 14.3 \times 10^6 \, mm^4$$

Finding I_{ef} where $M_s = M_u$

$$I_{ef} = I_{cr} + \left(I_g - I_{cr}\right)\left(\frac{M_{cr}}{M_u}\right)^3$$

$$= 14.3 + (66.6 - 14.3)\left(\frac{1.8}{7.52}\right)^3$$

$$= 15.0 \times 10^6\ mm^4$$

From this effective moment of Inertia, it is now possible to calculate the theoretical deflection value at ultimate loading:

Deflection at failure load F_u: $$\Delta_u = \frac{1}{48} \times \left(\frac{F_u \times L^3}{EI_{ef}}\right)$$

$$= \frac{1}{48} \times \left(\frac{13520 \times 2200^3}{25{,}000 \times 15.0 \times 10^6}\right)$$

$$\Delta = 8.0\ mm$$

3.2.2.6 Shear

Checking for the requirement of shear reinforcing of the pilot beam is as follows:

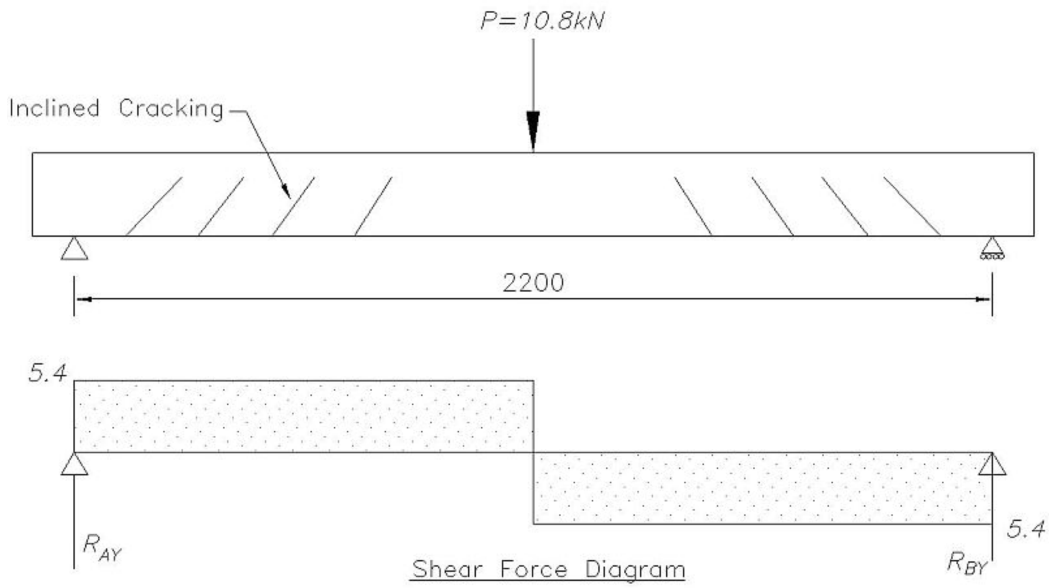


Figure 3.5 *Member shear force.*

AS3600 defines ultimate shear capacity as:

$$\nu_{uc} = \beta_1 \beta_2 \beta_3 \, b_v \, d_o \left(\frac{A_{st} \times f'_c}{b_v \times d_o} \right)^{\frac{1}{3}} \quad \text{where} \quad \beta_1 = 1.1 \left(1.6 - \frac{d_o}{1000} \right) \geq 1.1$$

$$\beta_2 = 1 + \left(\frac{N^{\times}}{14 A_g} \right)$$

$$\beta_3 = \left(2 \frac{d_o}{A_v} \right) \geq 2$$

$$\nu^{\times} \leq \phi V_u \; where \; \phi = 0.7$$

For project,

$$\nu^{\times} = 5.4 \, kN$$

Shear carried by concrete,

$$\beta_1 = 1.1 \left(1.6 - \frac{175}{1000} \right) = 1.567$$

$$\beta_2 = \beta_3 = 1.0$$

$$\nu_{uc} = 1.567 \times 1 \times 1 \times 100 \times 175 \left(\frac{71 \times 20}{100 \times 175} \right)^{\frac{1}{3}}$$

$$= 11.87\,kN$$

$$0.5\,\phi\,\nu_c = 0.5 \times 0.7 \times 11.87$$

$$= \mathbf{4.15\,kN} \quad < \quad \nu^{\times}$$

This suggests that shear reinforcing is required; however checks for minimum shear requirements are now valid.

Calculate $\nu_{u.min}$: $$V_{u,\min} = 11.87 + 0.6 \times 100 \times 175$$

$$= 22.37\,kN$$

Is $A_{sv.min}$ sufficient? $$\phi V_{u,\min} = 0.7 \times 22.37 = 15.7\,kN \quad > \nu^{\times}$$

$$\therefore \textit{no additional shear reinforcing required} \cdot$$

$$A_{sv,\min} = 0.35\,b_v \frac{s}{f_{sy \cdot f}}$$

$$\frac{s}{A_{s \cdot \min}} = \frac{f_{sy \cdot f}}{0.35 b_v}$$

$$\frac{s}{A_{s \cdot \min}} = \frac{500}{0.35 \times 100} = 14.3$$

$$\frac{A_{s \cdot \min}}{s} = 0.07$$

So from this, R6 @ 400crs shear reinforcing is considered acceptable. However it was decided that with the use of factor of safety modifier's, the use of shear reinforcing was to be excluded from beam construction.

3.2.3 Stepped Beam Design

The design of the stepped beams forms the heart of the project. It is the variations of reinforcing location of each that will influence the testing results, and form the basis of analytical dissection.

The prerequisites for the stepped beam design follow similar rules as to the pilot beam design; however it will feature a step in the horizontal axis central to the mid span of the member similar to Figure 2.2. The sectional size will follow that of the Pilot Beam (i.e. 100x200), however the lateral shape will comply with AS2870 shown in Fig 1.2.

Figure 3.6 details the step dimensions complying with AS2870's maximum step requirements.

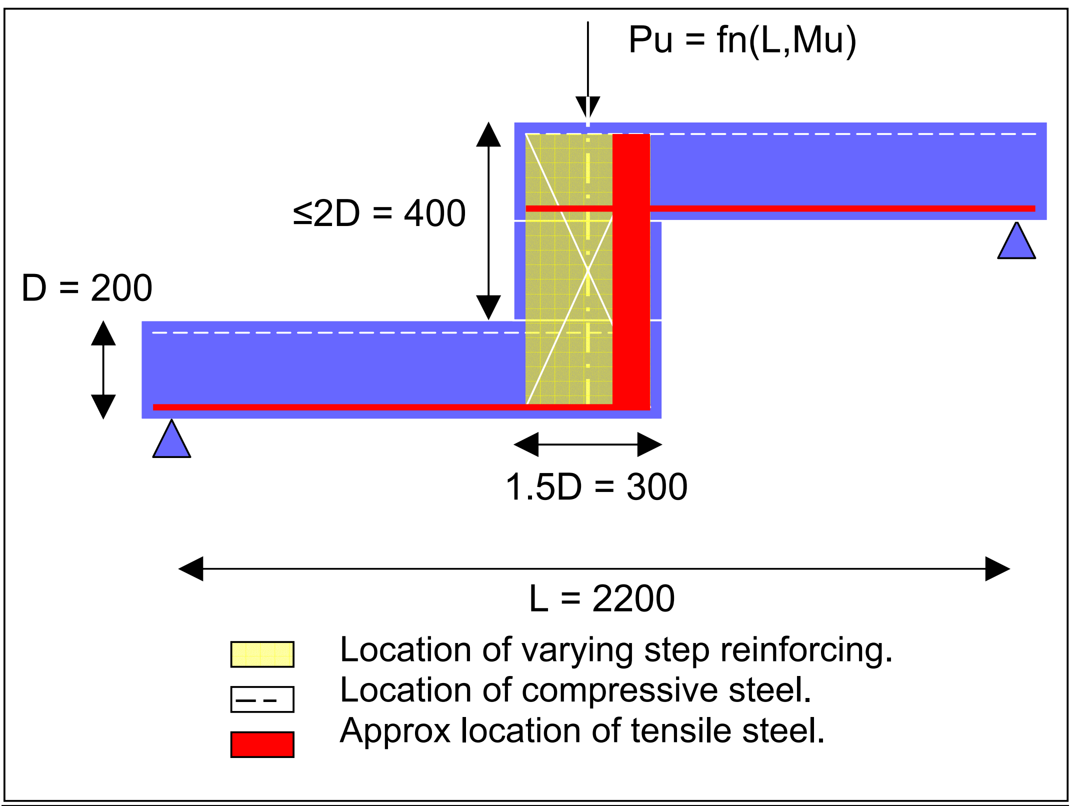


Figure 3.6 *Stepped beam design.*

From Figure 3.6, the locations marked red are to experience maximum tensile stresses, and therefore critical locations for steel reinforcing. The location marked yellow indicates the step ‘envelope’ and this region is where all changes in reinforcing steel will occur. Outside this area will remain consistent with the pilot beam design.

Four (4) examples of the stepped beam are to be constructed, and split into two (2) theoretical categories; Satisfactory and Unsatisfactory step envelope reinforcing arrangements (This assumption is based on basic engineering principles and professional footing exposure). It is also noted that although upper and lower member reinforcing steel is provided in all designs, the function of the upper or compression steel is predominantly seldom in the simply supported and one dimensional loading pattern.

3.2.3.1 Development Length Requirements

Section 13, Clause 13.1.2.1 of Australian Standard AS3600 – 2001 Concrete Structures provided the requirements for satisfactory reinforcing anchorage for steel in tension. This follows:

$$L_{sy \cdot t} = \frac{K_1 K_2 F_{sy} A_b}{\left(2a + d_b\right)\sqrt{f'_c}} \geq 25 K_1 d_b$$

Using 9.5mm Ø deformed wire,

$$L_{sy \cdot t} = \frac{1 \times 2.4 \times 500 \times 71}{(40 + 9.5)\sqrt{22.78}} \geq 25 \times 1 \times 9.5$$

$$= 360mm \geq 237mm$$

Therefore minimum development length of steel in tension is 360mm. Full explanation of modifying development length factors can be found in AS3600.

3.2.3.2 Satisfactory Design.

Satisfactory design for the stepped beam is only based on assumption at this stage. It is believed that the overall efficiency of these 2 beams when tested against the performance of the pilot beam will be within an acceptable range. Using engineering static principles and stress/strain limits and theories, it is believed that steel in tension requires an adequate development length past a maximum point of stress. In sections such as the stepped beam in Figure 3.6, size restraints require such development length of a tensile reinforcing bar to bend or 'turn the corner' in axial direction. This corner then leads to a vector of resultant force based on the theory of knee joints previously studied. With this theory in mind, the location of resultant forces must not be applied to areas of member already in tension. The adopted steel arrangement for both satisfactory stepped beams can be observed in Figure's 3.7.

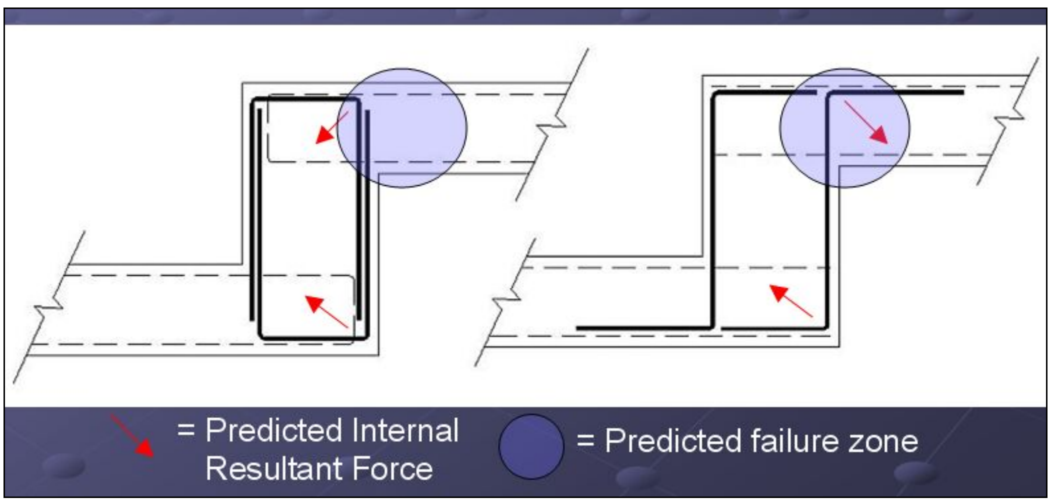


Figure 3.7 *Satisfactory beam design.*

Referencing back to Figure 3.7, the right hand vertical steel of the step envelope will be working in tension. It is the performance of this bar and its resultant force at the corner that will influence overall moment capacity.

3.2.3.2(A) Beam 2.

In beam 2 the use of simply constructed 'U' bars of 9.5mmØ RW is detailed. Compliance with 'ease of construction' and similar steel size to horizontal member reinforcing is achieved. Minimum development length is far exceeded by its vertical-horizontal-vertical shaping. Resultant forces acting at steel bends are directed to the core of the step and away from locations of tension. It is also noted that the reinforcing area of steel throughout the step envelope is also doubled due to the 'U' bar overlapping.

3.2.3.2(B) Beam 3.

Beam 3 assumes similar theory to Beam 2, however the development length of tensile reinforcing steel (retaining the assumption of downwards singular point loading) is mirrored away from the step core and along the remaining length of horizontal member. Although this bend and its resulting force is acting in a zone of compression, it is working toward area's of tension. The performance of this beam and its effects of resultant vectors will be observed during testing.

3.2.3.3 Unsatisfactory Design.

Similar to the satisfactory design for the stepped beams, the unsatisfactory beams are based on assumption and engineering principles. It is believed that the overall efficiency of these 2 beams when tested against the performance of the pilot beam will be far from an acceptable range. Again using engineering static principles and stress/strain limits and theories, it is believed that steel in tension will exhibit an inadequate development length past the maximum zone of stress. The adopted steel arrangement for both unsatisfactory stepped beams can be observed in Figure's 3.8.

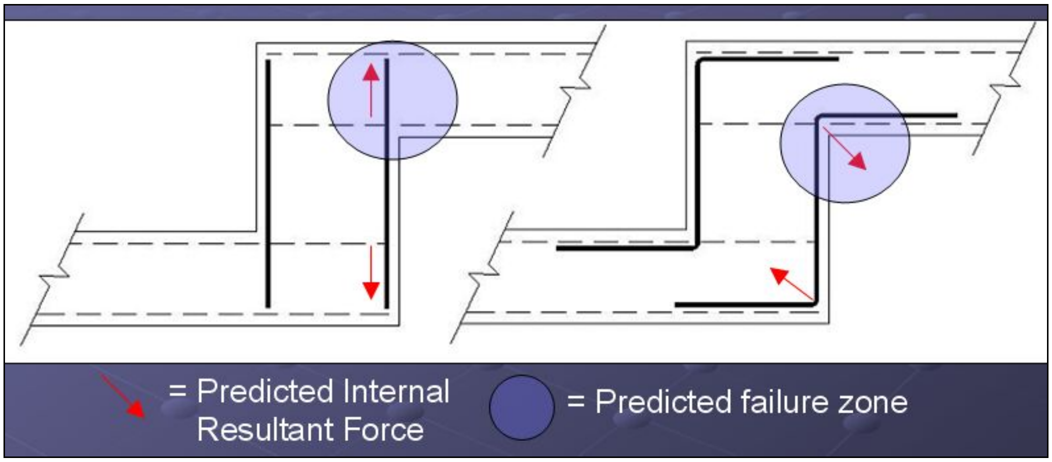


Figure 3.8 *Unsatisfactory beam design.*

3.2.3.3(A) Beam 4

Both of these step arrangements have been unfortunately witnessed on a professional engineering platform, however the understanding behind such is completely lacking. In Beam 4, again using 9.5mm RW reinforcing steel, the development length of this step is under design limits. With only 180mm of bar development past critical locations it is expected that this beam will develop friction failure between steel and concrete in this 180mm region far before maximum yield stress/moment capacity can be attained by the tensile reinforcing steel.

3.2.3.3(B) Beam 5

Beam 5 detailing is very similar to the assumed satisfactory Beam 3 arrangement. However in this example the bend of tensile steel reinforcing is located in a region of existing tensile forces. The actual development length criterion is satisfied, yet the resultant compression force at the steel bend is contributing now to the tensile forces in this point. It is believed that this beam will be the least efficient of all 4, and premature failure will occur very early in load testing to the upper underneath concrete step corner.

The variation of beam step arrangements is believed to result in very useful data to help understand the mechanics of structural concrete steps. By logging deflection vs. loading rates, it is anticipated that plastic/elastic, yield and ultimate values can be attained for each beam.

3.3 Beam Construction

The construction of the scaled concrete beams for testing followed the process of boxing, detail and placement of reinforcing steel and the pouring and curing of concrete. This process was conducted on a current industrial job site will the generous help from Steinmuller Constructions Pty Ltd.

3.3.1 Boxing

The beams were to be constructed on its 'y' strong axis to enable improved concrete placement and increased workability during steel reinforcing installation. Figure ___ displays this arrangement.

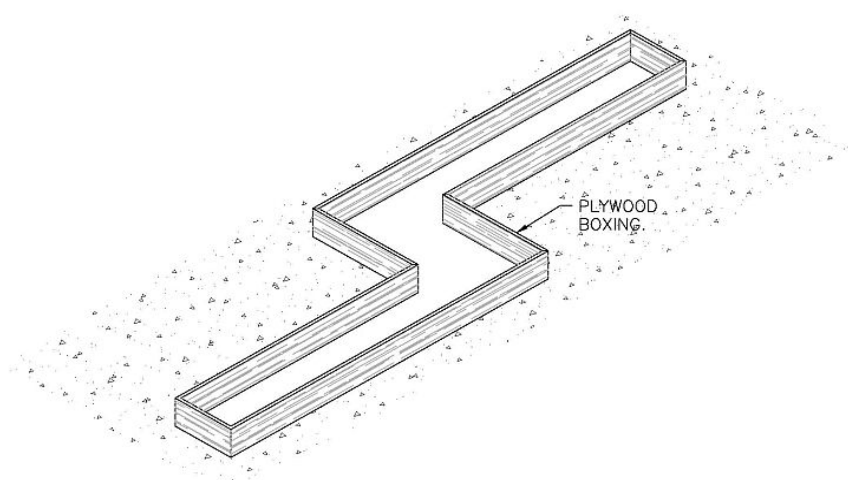


Figure 3.9 *Formwork arrangement.*

Formwork consisted of 3 ply sheeting, cut and shaped to exact design dimensions using power and hand tools. General timber screws were used for butting corners of formwork and a thin ply sheet was sized, cut and attached to the base of all formed moulds in order to offer 'cross-bracing' resistance to keep the moulds exactly square. During concrete placement and curing, it was anticipated that the formed plywood sides may 'bow' due to horizontal concrete pressure, therefore 70x35 stud timber bracing was also provided at multiple locations to the top of all formwork. The finished product of pilot beam and stepped beam formed moulds can be seen in Figure 3.10 below.

Figure 3.10 *Constructed formwork.*

3.3.2 Detailing

The 9.5mmØ RW reinforcing was sourced from local suppliers, at the amount of 7 x 6m lengths. From these lengths, each specific beam required custom lengths to be cut and shaped. Figure 3.11 shows the four (4) stepped beam reinforcing cut and shaped and ready for accurate placement within the step envelope. Wherever a bend occurs in a reinforcing bar stressed in tension, compressive forces are needed in the adjacent concrete to achieve the change in direction of the steel force. (Ragner, Hall). If the radius of the bend is to small, crushing of the concrete can occur due to an increased concentrated resultant force at the bend apex. The larger the radius of bend, the better distributed the resultant force as the bends resultant force is integrated around the larger bend. AS3600 Cl. 19.2.3.2 defines this minimum bend as 3_d or 3 times the diameter for wire steel, and 4_d for 400MPa and over steel. For this situation, 4_d or 34mm (8.5 x 4) was adopted as the minimum bend radius.

Figure 3.11 *Step envelope reinforcing.*

Such detailing was suspended within the section to exact layout (as per design requirements) with the aid of thin steel tie-wire (Figure 3.10). This tie-wire punctured the formwork with small drill holes and was to remain within the section during pouring and testing, and was of the scale that its presence would not influence any strength performance of the beam itself. Along with restraints in the vertical y axis plane, it was also necessary to provide similar restraint to the x axis to ensure all steel remained central to the section. This was achieved by using tie-wire suspended in a similar fashion through the top stud timber bracing shown in Figure 3.12.

Figure 3.12 *Reinforcing support and positioning.*

3.3.3 Pouring and Curing

Once all 5 beam moulds were accurately constructed and detailed, the placement of concrete could begin. Due to volume constraints, it was decided to not use the USQ concrete batching machine and instead place concrete at the construction site coinciding with a concrete foundation pour. This decision was based on the question of consistency of concrete strength if multiple concrete batches had to be made with the USQ machine.

N20 (20 MPa) concrete strength was placed in moulds by wheel barrow and shovel, hammer vibrated and then trowel finished. Curing then proceeded over the next 14 day period with boxing removed on day 7 and testing of beams on day 15.

To measure the 15 day concrete strength, a series of concrete test cylinders (6 x 100Ø, 2 x 150 Ø) were also poured from the same original beam concrete batch. These are to be tested at the University of Southern Queensland's laboratories in order to obtain average concrete compressive strengths for post testing calculations. Figure 3.13 shows the 7 day cured and unboxed concrete beams.

Figure 3.13 *Finalised beam construction.*

3.4 Conclusions: Chapter 3

The methodology detailed above has outlined the process behind design, construction and proposed testing methods. It has described the aims and objectives of how the testing is to be conducted, what approximate loads will be required and how we may expect the beams to behave. Figure 3.14 shows the adopted beam dimension size and span, while Figure 3.15 summarises the reinforcing details for each member.

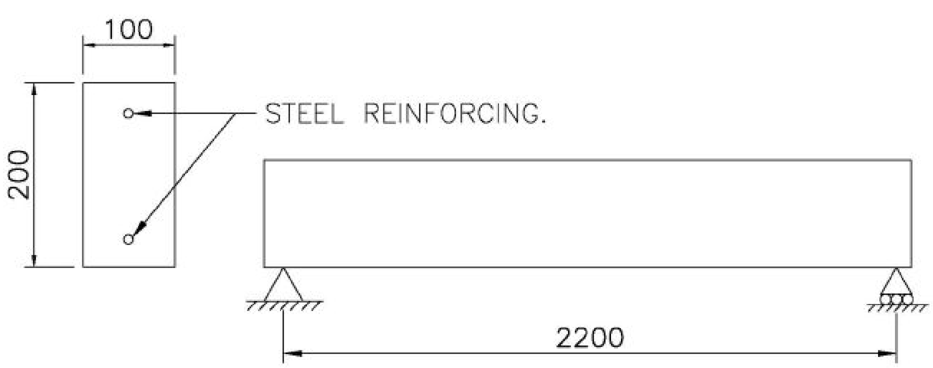


Figure 3.14 *Preliminary beam dimensions*

Leading on from design and construction, the beams will follow to testing at the University of Southern Queensland's laboratory, whereby each beam will undergo loading vs. deflection analysis and produce final graphed data.

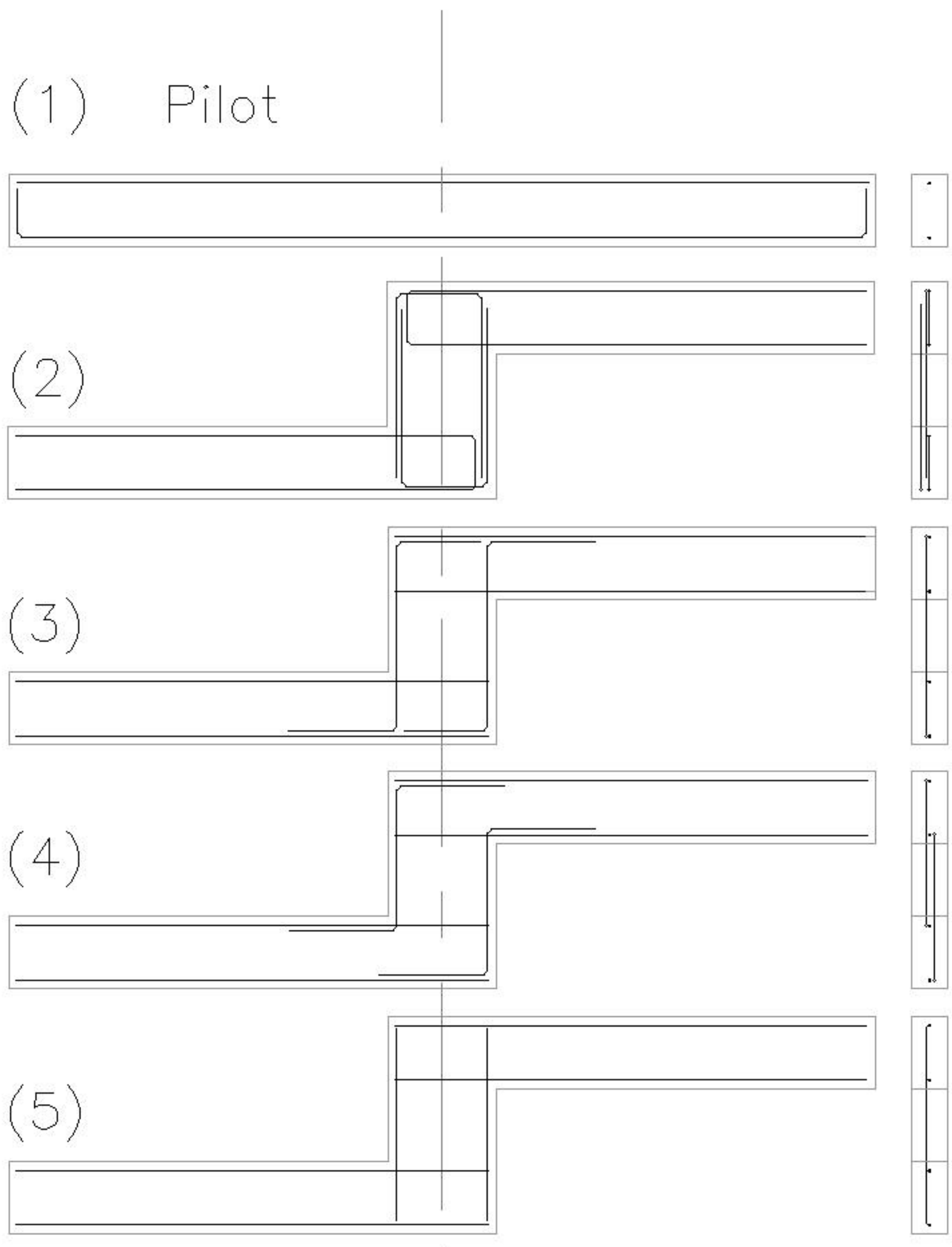


Figure 3.15 *Final beam arrangements.*

CHAPTER 4

MEASURED PERFORMANCE

4.1 Introduction

This chapter will detail test procedures of the 5 concrete beams, and the resulting performance of each including deflection vs. loading data and the formed crack patterns throughout testing. Compressive concrete test results will also be outlined to determine concrete member strengths at the time of loading. To follow, each beam's performance will be analysed individually, concluding with an overall efficiency comparison to the pilot beam.

4.2 Testing Apparatus

The testing apparatus for measuring each beams strength characteristics was performed by the University of Southern Queensland's Instron® Machine. Pictured below in Figure 4.1, this machine uses electronically controlled hydraulic pressure (capable of approx. 450kN of applied load) to provide either compressive or tensile force applications dependant on setup. As this testing was to follow a 3 point loading arrangement, the beam was placed centrally below the Instron machine, simply supported at both ends via adjustable height roller pins. The roller pins were to replicate a pinned joint connection as assumed in methodological assumptions.

In order to provide a point load to the beam as concentrated as possible, a steel plate of 20mm width was placed under the Instron pressure plate. This reduced bearing surface area from $0.015m^2$ to $0.002m^{2,}$ a decrease of over 750%.

Once all beam placement was compete, the Instron machine delivered the equivalent point load in deflection vs. time increments. This deflection is then digitally logged against comparative loading values to produce a final deflection vs. load graph for each beam. To follow, the beam's behaviour during loading will be observed, defining points of plastic and elastic regions (yield), ultimate loading, loading duration and the load causing final failure if applicable.

Figure 4.1 *Intron Testing Machine.*

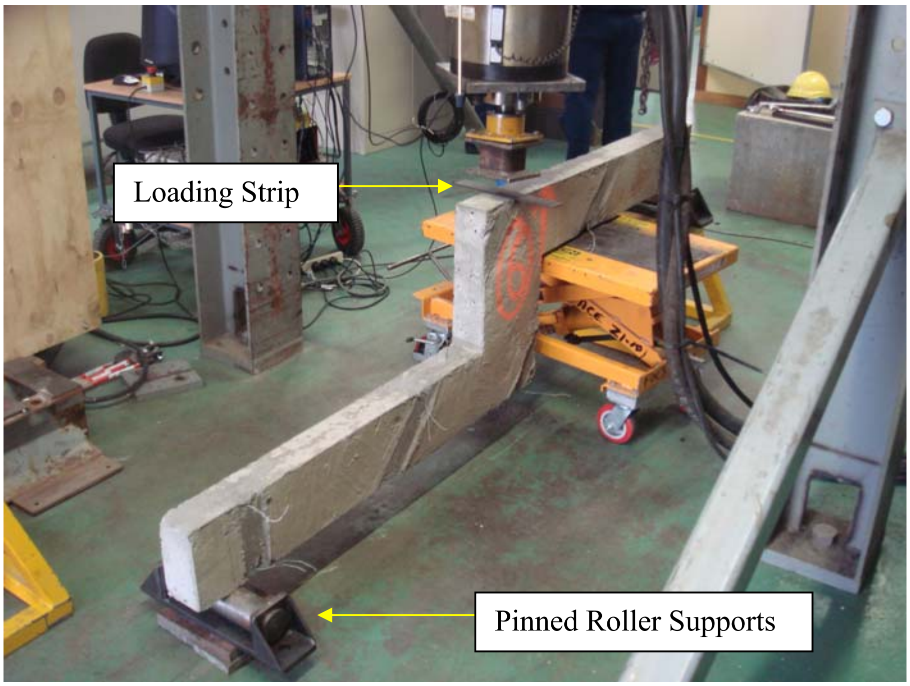


Figure 4.2 *Beam 6 setup for loading.*

4.3 Testing of Concrete Beams

Each beam was loaded to ultimate moment capacity, with loading extended on various beams to investigate crack pattern behaviour and failure modes. Completion of testing allowed moment capacity efficiencies to be defined.

4.3.1 Pilot Beam 1

The performance of the pilot beam was to be expected due to its simplistic design and purpose. Figure 4.3 (full size attached as Appendix B)shows it's deflection vs. load behaviour in which there are evident points of elastic and plastic performance, ultimate beam flexural capacity and then complete failure due to its extended loading time.

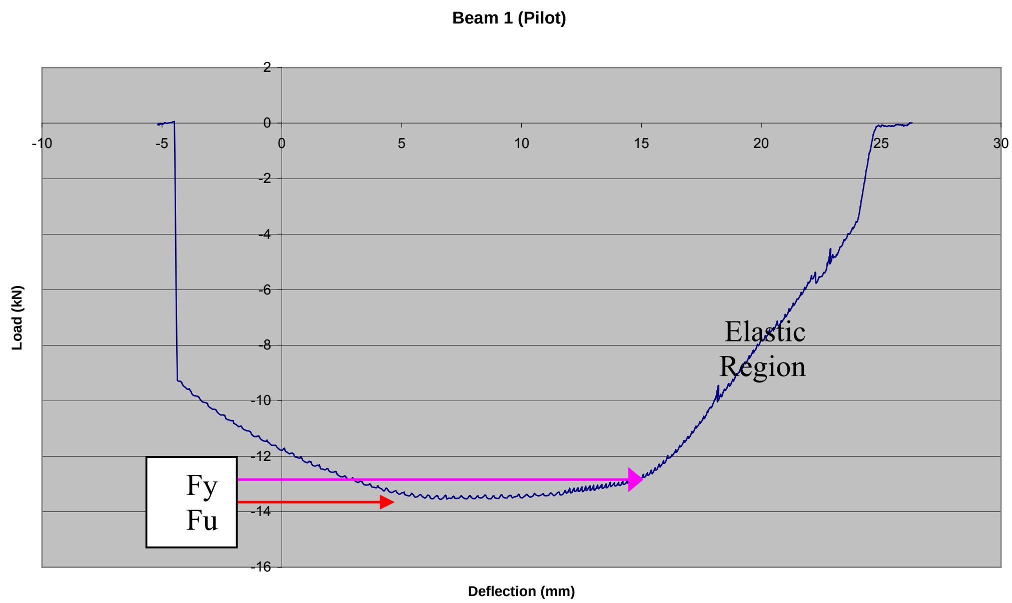


Figure 4.3 *Beam 1 Load vs. Deflection graph.*

Final performance data for the Pilot Beam is as follows:

$F_y = 12.56$ kN	$\Delta_y = 9$ mm
$F_u = 13.52$ kN	$\Delta_u = 15.75$ mm
$F_r = 9.25$ kN	$\Delta_r = 29$ mm

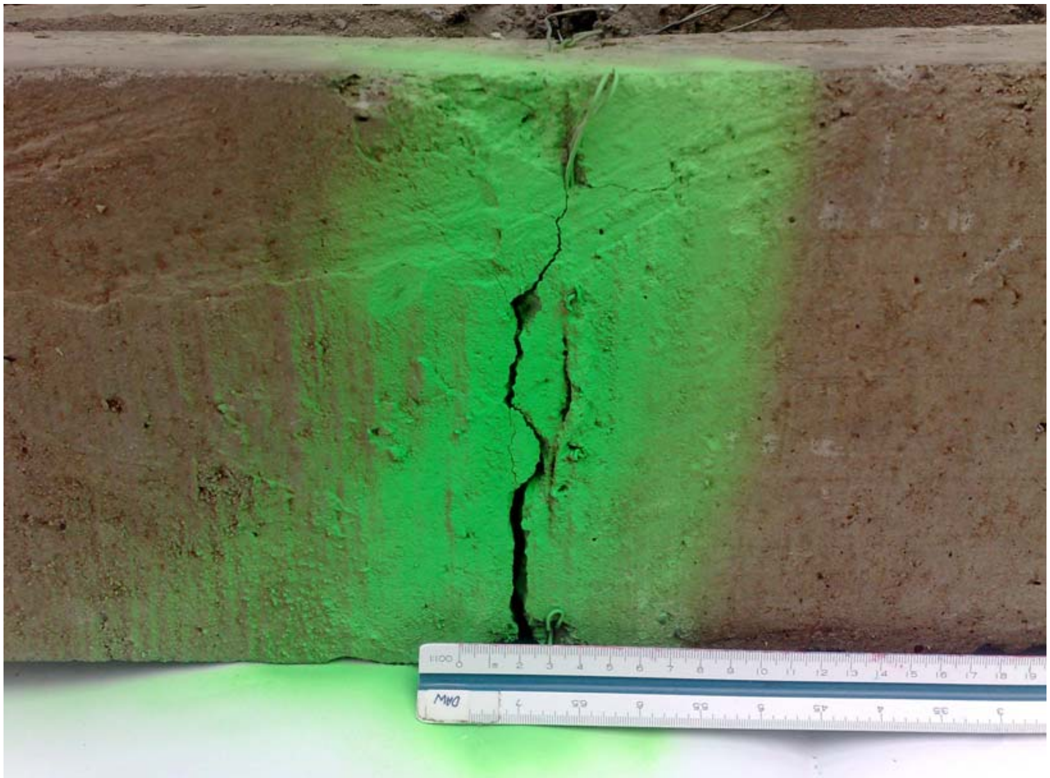


Figure 4.4 *Beam 1 crack patterns.*

4.3.1.1 Crack behaviour

Flexural cracking to the Pilot beam initiated at approximately 9-10kN of applied load and virtually centre of the supported beam width. Cracking then travelled to approximately 1/3 of beam depth at yielding (expected lower concrete compressive region) and was seen above in Figure 4.4 to develop to d_c (depth of steel in compression). It was also observed that no shear cracking was evident during entire testing.

4.3.2 Stepped Beam 2

Beam 2 was the first of the stepped beam arrangements. Referring back to the Beam summary in Figure 3.15, this was a predicted satisfactory steel reinforcing arrangement. It's deflection vs. load performance can be observed in Figure 4.5, again noting yield, ultimate and rupture points.

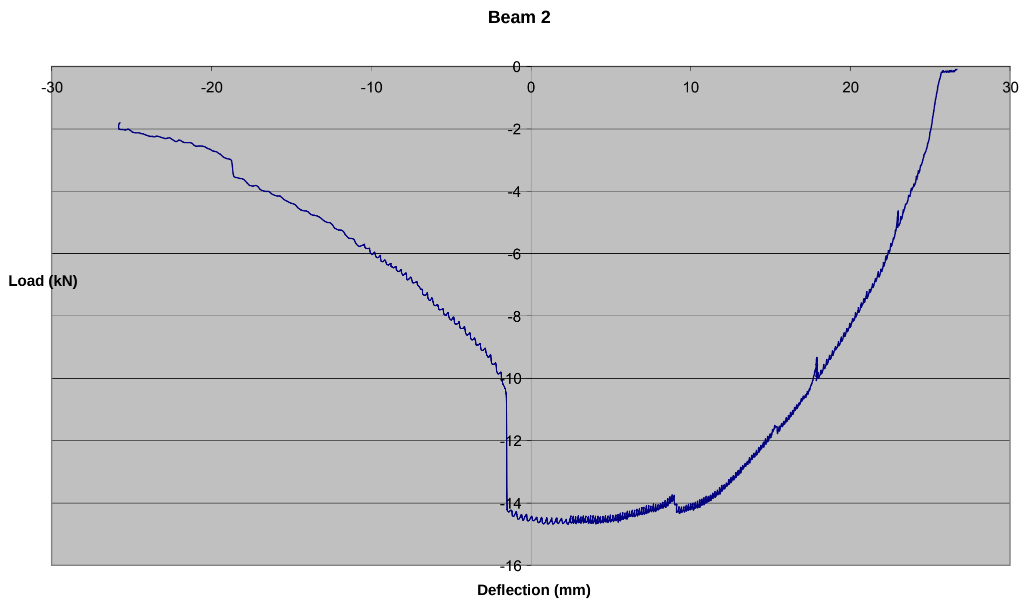


Figure 4.5 *Beam 2 Load vs. Deflection.*

Final performance data for Beam 2 is as follows:

$F_y = 13.68$ kN $\Delta_y = 14.9$ mm

$F_u = 14.62$ kN $\Delta_u = 23.9$ mm

$F_r = 14.20$ kN $\Delta_r = 27.1$ mm

Figure 4.6 *Beam 2 crack patterns.*

4.3.2.1 Crack behaviour

Flexural cracking was observed at 5kN of applied load to the lower inside corner of the ‘opening’ knee joint. (Refer Figure 4.6). From this point hairline cracking proceeded vertically to the lower compressive concrete/steel zone. At 11.7kN applied load, integrity of the step envelope was maintained and initiation of hairline flexural cracking to the lower ‘closing’ knee joint was observed. Further loading however, was controlled by closing joint reinforcing, yet development of opening joint cracking continued. This continuation was seen to ‘scatter’ throughout the upper joint envelope as per Figure 4.6 and final rupture was due to concrete crushing within the upper opening knee joint step envelope.

4.3.3 Stepped Beam 3

Beam 3 was the second of the stepped beams and the last of the assumed satisfactory arrangements. It's deflection vs. load performance can be observed in Figure 4.7, this time its clarity between yield and ultimate loads somewhat vague. Load to rupture is not evident due to loading only taken sufficiently past ultimate.

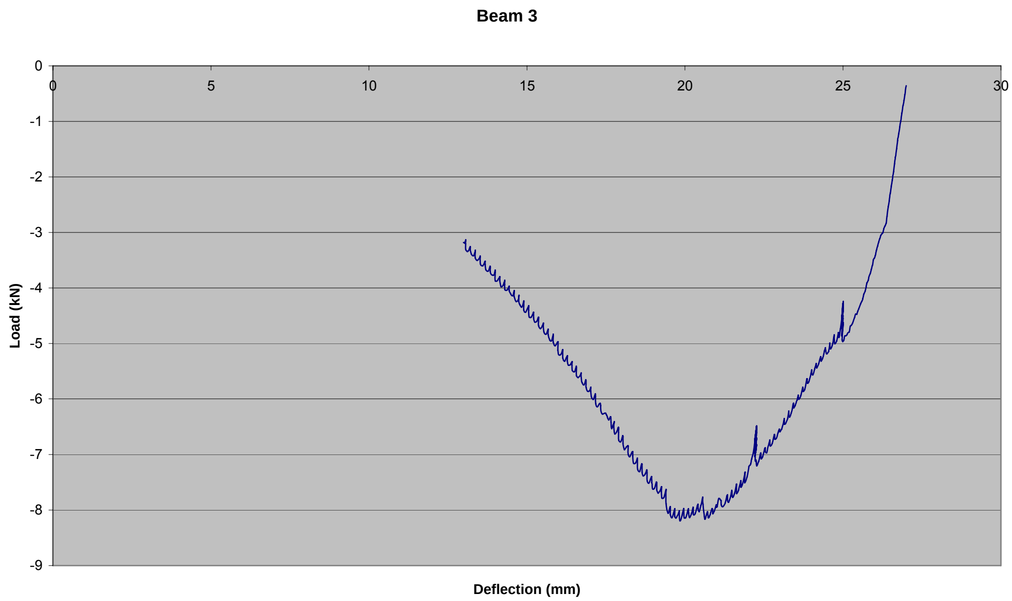


Figure 4.7 *Beam 3 Load vs. Deflection.*

Final performance data for Beam 3 is as follows:

$F_y = 8.16$ kN	$\Delta_y = 6.4$ mm
$F_u = 8.2$ kN	$\Delta_u = 7.2$ mm
F_r = na	Δ_r = na

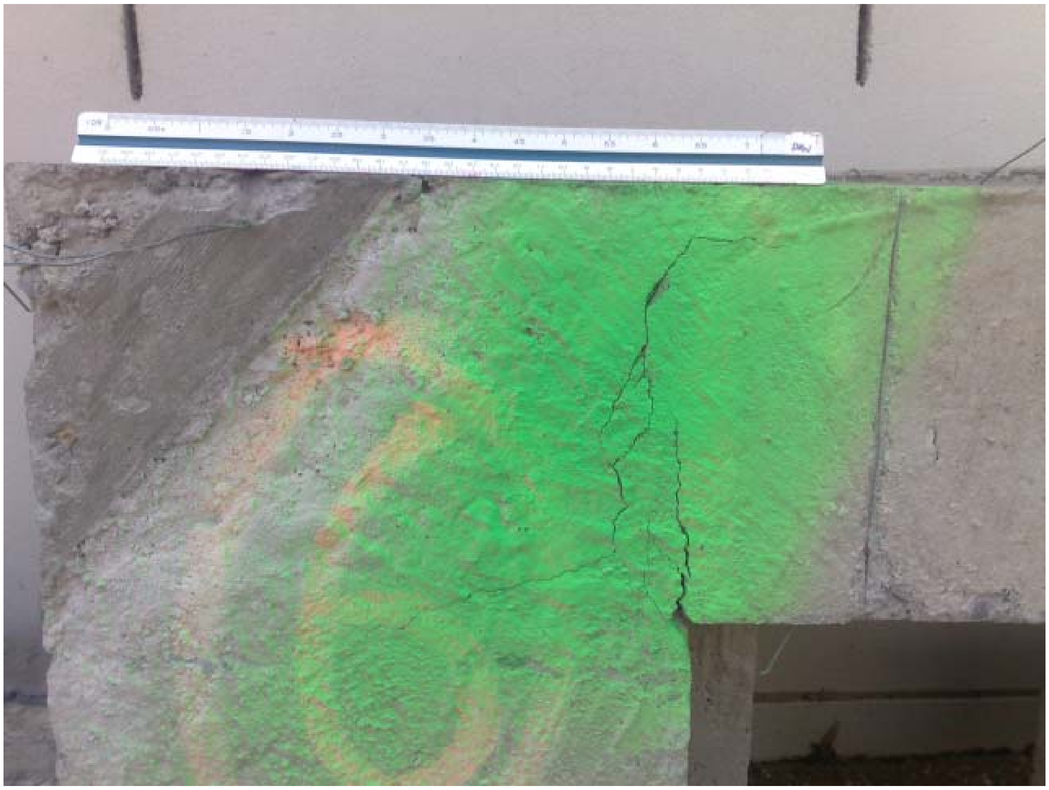


Figure 4.8 *Beam 3 crack patterns.*

4.3.3.1 Crack behaviour

Initial cracking developed at 6-7kN's applied load. This was observed (Figure 4.8) at upper and lower knee joints simultaneously, adjacent to inside joint corners, but perpendicular to adjoining horizontal members. At 8kN the crack path to the upper opening knee joint propagated horizontally across the step envelope terminating at compressive concrete zones. Similarly the opening knee joint vertical crack path extended to the upper compressive steel location, whereby it then proceeded horizontally away from the step envelope. As previously stated the crack widths were not to the extent of Beam's 1 and 2 due to ceasing loading soon after ultimate.

4.3.4 Stepped Beam 4

Beam 4 was the third of the stepped beams and the first of the assumed unsatisfactory arrangements. It's deflection vs. load performance can be observed in Figure 4.9, and similar to Beam 3 but far more extensive, its clarity between yield, ultimate and now also rupture loads are not visually distinguishable.

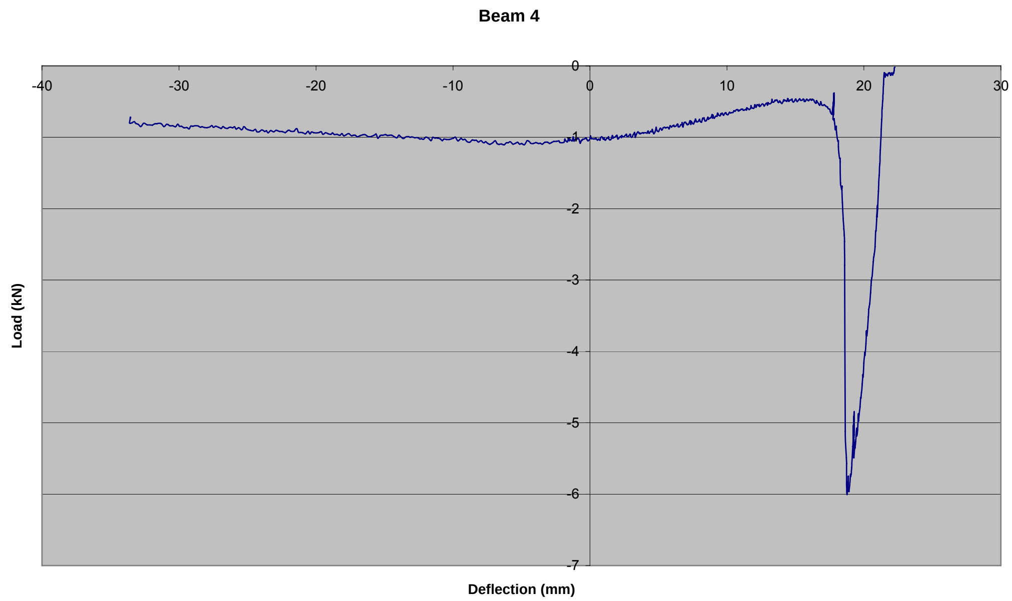


Figure 4.9 *Beam 4 Load vs. Deflection.*

Final performance data for Beam 4 is as follows:

F_y = na	Δ_y = na
F_u = 6.0 kN	Δ_u = 2.63 mm
F_r = 6.0 kN	Δ_r = 2.63 mm

Figure 4.10 *Beam 4 crack patterns.*

4.3.4.1 Crack behaviour

Typical to the other beams, cracking did not develop until around the 4-5kN load. Shortly after cracking moment as applied, extensive horizontal cracking was produced along the entire opening knee joints inside corner. This is clearly evident in Figure 4.10. Within less than 1kN of additional applied load, the beam had undergone brittle concrete tensile failure. The adjoining horizontal member to the opening knee joint corner then proceeded to exhibit concrete ‘dropping-out’ with extended loading and deflection.

4.3.5 Stepped Beam 5

Beam 5 was the fourth of the stepped beams and the second of the assumed unsatisfactory arrangements. It's deflection vs. load performance can be observed in Figure 4.11.

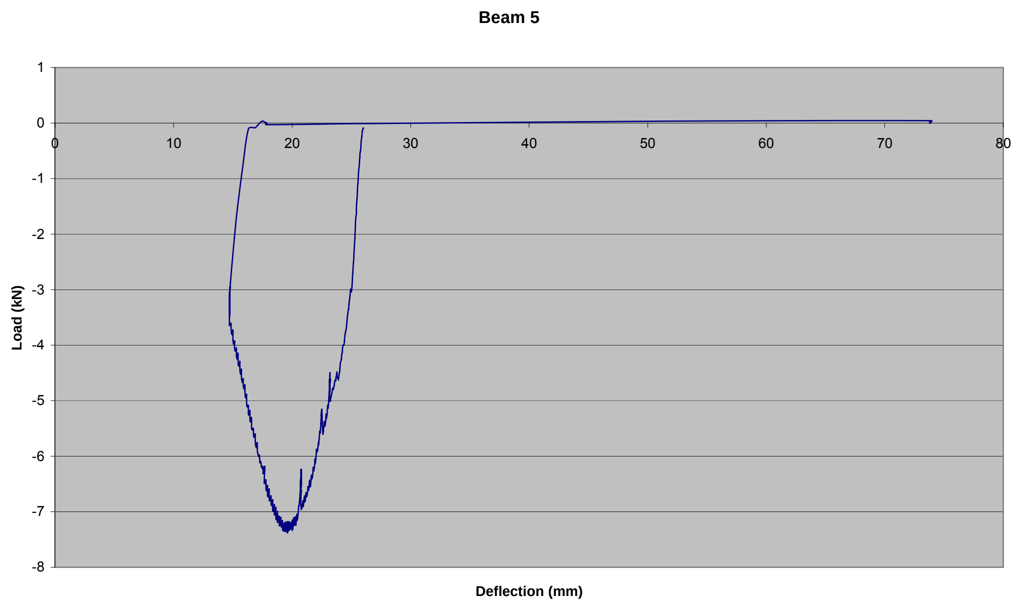


Figure 4.11 *Beam 5 Load vs. Deflection.*

Final performance data for Beam 4 is as follows:

F_y = na	Δ_y = na
F_u = 7.35 kN	Δ_u = 6.55 mm
F_r = na	Δ_r = na

Figure 4.12 *Beam 5 crack patterns.*

4.3.5.1 Crack behaviour

Again there was no cracking evident until after cracking moments applied (in excess of 5kN). At 6-7kN loading, typical upper knee joint inside corner 45 deg vertical cracking began. Cracking then proceeded both horizontally and vertically after it had met the tensile steel reinforcing within the step envelope. Full cracking in the vertical axis to compressive steel, and cracking to half the step width or approximate compressive concrete zones in the horizontal plane.

4.4 Concrete Compressive Testing

Along with beam testing, 6 concrete test cylinders were poured, cured and tested to determine the step beams compressive concrete strength at 15 days (time from pouring to testing). Design concrete strength of N20 is the Australian standard 28 day compressive strength. The change in Strength vs. Time is found to be very small from this point onwards. However since testing was at nearly half this time frame, it is necessary to determine its gained 15 day strength for result analysis. Although the cylinders were not tested until day 16, the increase in overall strength was considered to be marginal.

Testing comprised 4 x 100Ø x 200mm cylinders and 2 x 150Ø x 300mm cylinders for wider averaged strength properties. The following Table 4.1 lists tested results of the 15 day compressive concrete strength. Figures are based on the $\sigma = \dfrac{F}{A}$ equation.

Concrete Compressive Strength Tests

Sample (100Ø)	Ø (mm)	L (mm)	Load (kN)		Area (mm2)	F'c (MPa)
1	99.7	200	170		7806.70	21.8
2	100.3	199	178		7900.94	22.5
3	99.8	200	180		7822.37	23.0
4	99.5	200	183		7775.41	23.5
(200Ø)						
5	151.18	299	409		17950.05	22.8
6	152.27	302	420		18209.82	23.1
					Σ	136.7
					Avg F'c	**22.78**

Table 4.1 *Concrete compressive strength data.*

4.5 Conclusions: Chapter 4

The following Table 4.2 and graph in Figure 4.14 summarises the accumulated beam testing characteristics in regards to varying cracking, yield, ultimate and rupture loads. It will also outline the corresponding deflection values and observed modes of failure.

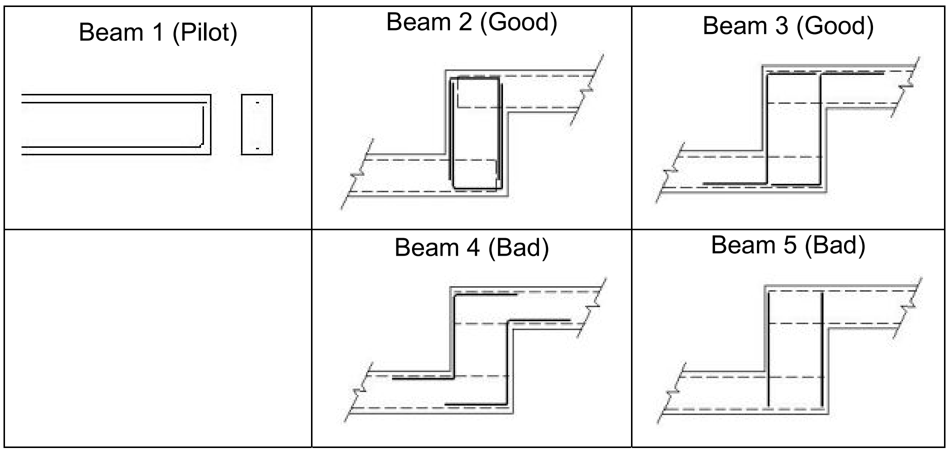


Figure 4.13 *Beam schedule.*

Beam Performance Characteristics

Beam	Observed Failure Mode	Flexure (kN)			Efficiency	Deflection (mm)		
		Fy	Fu	Fr	(%)	Δy	Δu	Δr
1	Tensile Steel Yield	12.56	13.52	9.25	100	9	15.75	29
2	Concrete Crushing	13.68	14.62	14.2	**109**	14.9	23.9	27.1
3	Friction	8.16	8.2	-	65	6.4	7.2	-
4	Tensile Concrete Yield	-	6	6	**47**	-	2.63	2.63
5	Friction	-	7.35	-	58	-	6.55	-

Table 4.2 *Summarised beam performance data.*

The performance of nearly all beams in testing for flexural strength could be considered as ‘expected’. However the behaviour and ultimate failure loading of Beam 3 would suggest there is room for investigation and/or improvement. The remaining beams (including the Pilot Beam) all performed well and failed in the modes predicted.

The published results do however show a remarkable spread of performance in ultimate moment capacity and corresponding deflection performance characteristics. These results can now be used for the basis of analytical calculations in order to further understand the performance of such knee joint or structural concrete step mechanics, and potentially offer a much greater insight into the proficient design and construction of such in real world situations.

(Full size load vs. deflection plots for each beam and Beam 1 raw test data can be found attached as Appendix B)

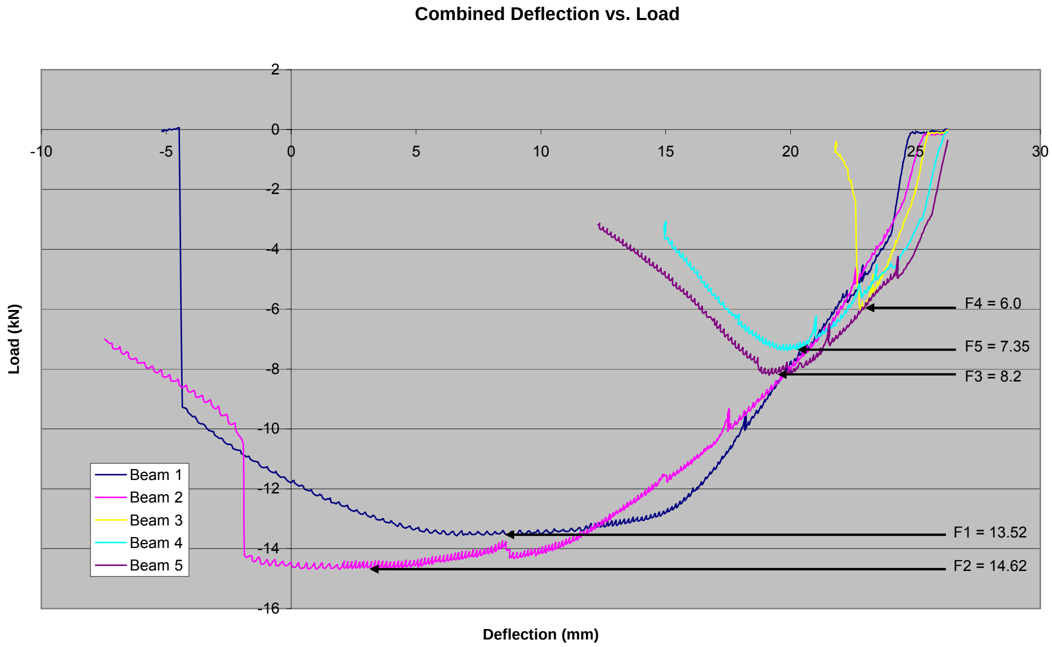


Figure 4.14 *Combined loading behaviour (Full size plot in Appendix #).*

CHAPTER 5

INTERPRETATION OF RESULTS

5.1 Introduction

Results of testing previously outlined, have demonstrated a vast range of performance data for each beam and its differing steel reinforcing arrangements. In this chapter, the individual behaviour and operation of such beams will be critically and singularly analysed in order to gain insight of the beams limiting variables and/or notable advantages. Using such findings, the design and construction of structural concrete steps using data from this research, will hopefully be commissioned to future real world situations.

From Figure 5.1 and Appendix B, it is observed that the behaviour of each beam exhibits similar load vs. deflection up to 6.0 kN of applied load. From previous calculations, cracking moment was expected at 1.8 kN.m or 3.27 kN applied point load. Observing load data, each beam performs in similar fashion up to concrete cracking moment. From this point the steel reinforcing is commissioned, and each beam's behaviour becomes exclusive. It is this post cracking behaviour that will be analysed to follow.

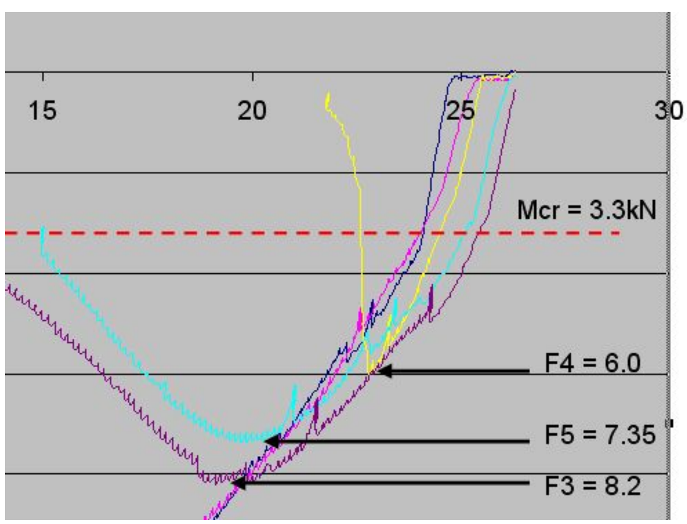


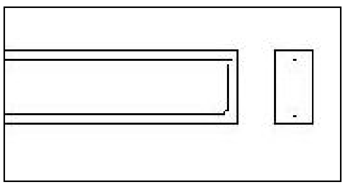
Figure 5.1 *Pre/post Cracking Moment behaviour.*

5.1.1 Ultimate load

It has been noted previously that the objective of this research to compare and improve various reinforcing arrangements for structural concrete beam steps. By using data obtained from load vs. deflection testing, the analysis of contributing factors for yield and ultimate loading will be undertaken. By understanding these factors, an educated theory of better performance may be achievable.

5.2 Beam Performance/Failure Analysis

5.2.1 Beam 1



5.2.1.1 Strength

The performance of Beam 1 otherwise known as the 'pilot beam' was expected to behave in the anticipated fashion. From previous design calculations it was predicted load to yield would occur at approximately 10-11kN applied point load, and a cracking moment of around 3.3 kN.

Development of the deflection vs. load graph confirmed that the beam under went a change in gradient (change in behaviour) at approximate 3.5kN loading. This cracking moment was also confirmed with hairline crack development at this loading. From this it is safe to assume that this change in gradient was infact the transition of tensile resistance from concrete to steel.

Continual loading progressed the flexural cracking vertically upwards of the beams mid-span location as loading increased. At 12.5 kN applied load (6.88kN.m), steel reinforcing transformed from elastic to plastic deformation. From this point onwards 'necking' of the transverse steel reinforcing began. A further 1 kN of applied load was gained before ultimate load was reached. This ultimate load was directly linked to maximum tensile stress development of the 'ribbed wire' reinforcing bar.

Final ultimate loading of 13.5 kN was very similar to the theoretical ultimate load capacity of 13.67 kN! It's yield point of 12.5 kN was greater then that of the predicted load, however this assumption was previously based on the minimum yield stress of 500 MPa being adopted. The following calculations verify the actual yield and ultimate stress exhibited by the reinforcing steel.

Yield (actual),

$$M_y = \left(71 \times \sigma_{st \cdot y} \times 175\right) - \left(38.53 \times \left(\frac{19.86}{2}\right)\right) + (3 \times 25) = 6.875\ kNm$$

$$\sigma_{st \cdot y} = \mathbf{578}\ \boldsymbol{MPa}$$

Ultimate (actual),

$$M_u = \left(71 \times \sigma_{st \cdot u} \times 175\right) - \left(38.53 \times \left(\frac{19.86}{2}\right)\right) + (3 \times 25) = 7.425\ kNm$$

$$\sigma_{st \cdot u} = \mathbf{622}\ \boldsymbol{MPa}$$

Using the sum of moments about the top fibre and concrete stress block and neutral axis depths as previously found, the calculations above show an increase in actual yield stress to the listed stress from the manufacturer. (a common conservative listing). Pending the similar placement of reinforcing steel in the section, it can be assumed that similar yield and ultimate steel stresses may be adopted for the other stepped beams in service.

It was also observed that small inconsistencies in load vs. deflection were apparent during elastic loading. It is believed that these variations in linear elastic behaviour are attributed to friction losses between the steel and concrete adhesion during loading.

5.2.1.2 Deflection

Deflection at ultimate load was predicted at 8.0mm. Testing deflection was measured at 15.7mm. This shows a considerable difference in estimated and tested deflection. Rangan (1985) suggests that for low reinforcement ratios

(<0.005), the tension stiffening effect may be greatly overestimated, which in turn leads to over-estimation of the effective moment of inertia (I_{ef}) and hence under-estimation of deflection. To counteract this deficiency, an upper limit of 0.6 I_g on I_{ef} has been proposed. Re-evaluating the original predicted deflection we obtain:

Finding I_{ef} where Ig = 0.6 Ig

$$I_{ef} = I_{cr} + \left(0.6 I_g - I_{cr}\right)\left(\frac{M_{cr}}{M_u}\right)^3$$

$$= 14.3 + \left((0.6 \times 66.6) - 14.3\right)\left(\frac{1.8}{7.52}\right)^3$$

$$= 14.6 \times 10^6 \, mm^4$$

From this effective moment of Inertia, it is now possible to calculate the theoretical deflection value at ultimate loading:

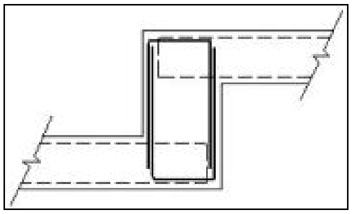
Deflection at failure load F_u: $$\Delta_u = \frac{1}{48} \times \left(\frac{F_u \times L^3}{EI_{ef}}\right)$$

$$= \frac{1}{48} \times \left(\frac{13520 \times 2200^3}{23{,}500 \times 14.6 \times 10^6}\right)$$

$$\Delta = 8.75 \, mm$$

Still after the reduction of the gross sectional inertia, final deflection predictions did not substantially alter. It is now believed that calculation of the cracking inertia is incorrect due to the very high depth of neutral axis combined with the upper ‘compressive’ steel acting in tension up to yield. Although the deflection predictions did not match actual, the strength of beam in flexure data did. It is the accuracy of this testing that will verify further testing analysis.

5.2.2 Beam 2



5.2.2.1 Strength

The performance of Beam 2 was considered the best in respect to ultimate moment capacity and performance. Based on the pilot beams previous performance, its testing was to reach 13.5 kN applied load to gain 100% efficiency within the step envelope. From data obtained from load testing, this stepped beam arrangement demonstrated a final ultimate efficiency of 14.62 kN. This is 109 % efficiency when compared to the ultimate load performance of the pilot beam. This result shows exceptional structural integrity of the step envelope and the reinforcing steel within.

From previous theory on the resultant force's magnitude and direction, it was shown from crack patterns in Figure 4.6, that these forces were not isolated to a single point in the beam, especially a single point in a tensile zone. Final failure of this beam was concluded by concrete crushing in the upper knee joint step envelope. Upon concrete crushing to this region, the upper horizontal member reinforcing began to lose friction or development length and in turn rupture occurred from this point on. This was confirmed during later inspection of the stepped beam when complete separation occurred between the upper member and step envelope as seen in Figure 5.2. The steel reinforcing to both items was still intact and showed no apparent signs of steel yielding or 'necking'.

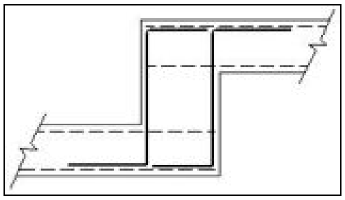
Flexural performance of the stepped beam also confirmed area's of compression and the advanced operation of the 'closing' knee joint as previously defined.

Figure 5.2 *Destruction loading results to Beam 2.*

5.2.2.2 Deflection

Although no specific deflection calculations were undertake prior to stepped beam testing, its behaviour in deflection can also be related back to the pilot beams deflection path. As shown in Figure 5.1, from 8 kN to 12 kN applied loading, the deflection performance of this beam lacks up to 20% efficiency in serviceability to that of the pilot beam. It's continually smooth deflection path also indicates the lack of characteristic yield point. This also backs up the conclusion of concrete crushing as the mode causing failure.

5.2.3 Beam 3



5.2.3.1 Strength

The performance of this arrangement was predicted to follow a good efficiency. In other words it was predicted to be closely matched to the pilot beams flexural capacity and deflection behaviour. Its ultimate load application occurred at 8.2 kN applied load, an efficiency of only 65% of the pilot beams. In this failure, it was observed that friction or lack of development length was the main cause of failure. From previous calculations, the required development length for the 9.5mm Ø Ribbed Wire reinforcing was 360mm. In this design, although the step envelope reinforcing steel was believed to be satisfactory, it is now clear from testing that the horizontal upper steel did not comply with this condition and it was indeed this lack of friction that caused failure at such an early stage. This influencing factor can be seen in Figure 5.3.

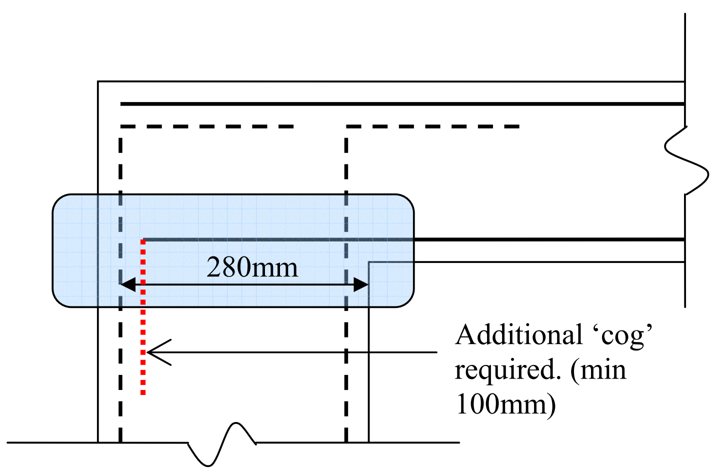


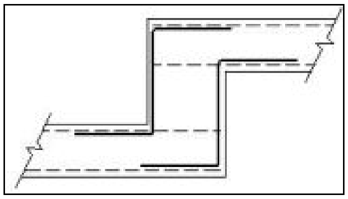
Figure 5.3 *Failure mode diagram.*

Referring back to Figure 4.14, early performance of the beam was similar throughout uncracked loading, and also provided similar elastic behaviour up to the premature ultimate failure. A distinct (but not sudden) reduction in force followed ultimate failure as a result of this gradual but constant increase in frictional loss. It is observed that a substantial increase in efficiency would be gained by providing a ‘hook end’ or ‘cog’ of 100mm minimum to the tensile reinforcing of the upper adjoining horizontal member as shown in Figure 3.7. This alteration would satisfy the minimum development length required for steel in tensile operation. This arranged is based exclusively to the step dimensions adopted for this research. A wider step then 1.5D (as per AS2870) may overcome lack of development length dependant on other factors such as section and reinforcing steel sizes.

5.2.3.2 Deflection

The deflection characteristics of this beam followed pre concrete cracking similar to the pilot beam. Post cracking moment also exhibited identical elastic deflection behaviour up until premature failure occurred. The deflection characteristics appeared satisfactory however this assumption is void in service due to its unsatisfactory flexural capacity.

5.2.4 Beam 4



5.2.4.1 Strength

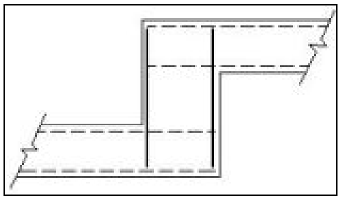
This stepped arrangement was tested to be the worst in both flexural efficiency and deflection limits combined. An ultimate load of only 6 kN meant this arrangement only achieved 47% of the pilot beams strength. As was the case with the previous stepped beams, pre cracking moment behaviour was shown to be similar, yet its shortened service life after cracking moment disabled this beams ability to provide any further performance characteristics. Although the horizontal member's tensile steel had a comparable design to that of Beam 3 (i.e. lack of development length), it was the predicted resultant force of the opening knee joint's inner corner reinforcing that caused failure. After cracking moment of 3.3kN, the step envelope's reinforcing steel was placed in service, instantly becoming tensile and trying to straighten at the upper inside corner. This compressive resultant force was distributed instantly to a section of beam already in tension, effectively adding to this tensile concrete/steel force.

This crack development propagated horizontally across the step envelope, placing the horizontal tensile steel into a reduced frictional environment. From this point the opening knee joint became predominantly unreinforced in all tensile regions. The advancement to 6kN maximum applied load can be attributed to marginal retention of concrete to steel friction along the step envelope's reinforcing 'cog'. This example has 'reinforced' the assumption that this arrangement is critically deficient and any attempt to reconstruct a step with this arrangement should be completely abandoned.

5.2.4.2 Deflection

As per Beam 3, the deflection characteristics of this beam followed pre concrete cracking similar to the pilot beam. Post cracking moment also exhibited identical elastic deflection behaviour up until premature failure occurred. The deflection characteristics appeared satisfactory however this assumption is void in service due to its unsatisfactory flexural capacity.

5.2.5 Beam 5



5.2.5.1 Strength

This stepped configuration was the last beam tested, and represented the second unsatisfactory arrangement. At just under 60% efficiency it was not the worst, but still far from acceptable compared back to the pilot beam. Its observed failure mode was loss of concrete to steel friction due to inadequate steel development length. Although similar in failure to Beam 3, this arrangement displayed 2 examples of lacking development length in locations of tensile stress. Figure 5.4 shows these 2 locations which also coincide with observed concrete cracking.

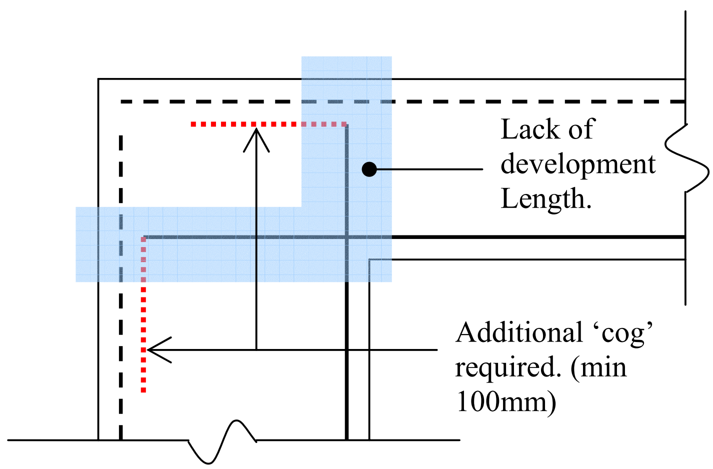


Figure 5.4 *Pre/post Cracking Moment behaviour.*

The decreased efficiency of this beam is concluded to be a combined effect of the loss of friction in both the x and y axis's in tension. In Beam 3 its horizontal reinforcing was sufficiently developed due to its upper member 'cog'. In this situation, development past the point of maximum tensile stress is only 160mm vertically and 280mm horizontally. This combination has coincided with one another to result in ultimate moment capacity fractionally lower then Beam 3. Observation of the load vs deflection graph indicates moments of abnormality throughout elastic behaviour that indicates frictional losses to the steel reinforcing under tensile stress.

Remedial design of this arrangement can be made to improve efficiency by the extension of development length as shown by the 'dotted' lines in Figure 4.18. This can be achieved by 'cogging' main reinforcing or 'splicing' with minimum lap lengths.

5.2.5.2 Deflection

As per Beam 3, the deflection characteristics of this beam followed pre-concrete cracking similar to the pilot beam. Post cracking moment also exhibited identical elastic deflection behaviour up until premature failure occurred. The deflection characteristics appeared satisfactory however this assumption is void in service due to its unsatisfactory flexural capacity.

5.3 AS2870 beam step constraints

The step dimensions taken from AS 2870 – Residential Footing and Slabs, has limited the step height to two (2) times the depth of footing (Refer Figure 5.5). Although it has been concluded that the overall step height does not play the limiting factor in step design, it is the notation of reinforcing steel's development length that has reason for concern.

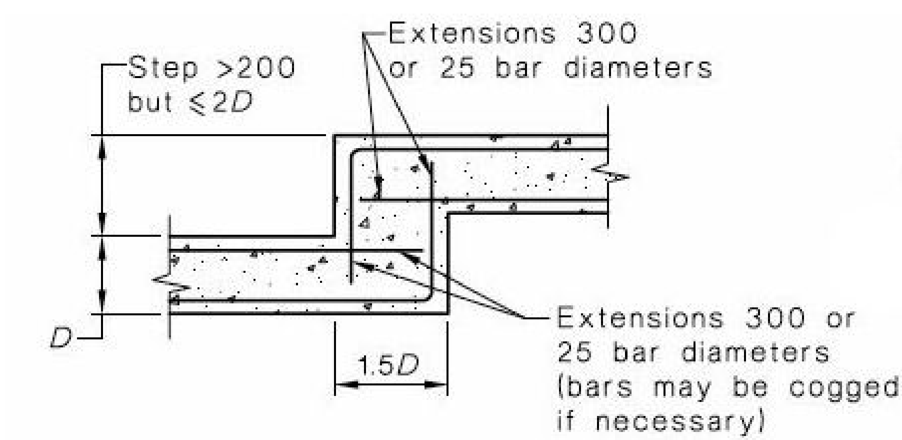


Figure 5.5 *AS2870 Beam step diagram.*

Its reference to 300mm or 25 bar diameters does not clearly define its minimum requirements. Using such limits for the beam sizes tested, this allows development lengths of either 300mm or 237mm (9.5mmØ x 25), which has been proven from test data to be far unacceptable values. Even refining foundation depth ‘D’ and the steel reinforcing size, in most real world foundation sections, these Australian Standard limitations become far more critical.

5.4 Conclusions: Chapter 5

Testing data has outlined large differences in the efficiency of stepped structural concrete beams to maintain the moment continuity along the member. The variety of steel reinforcing between the 5 constructed beams have displayed valuable results in the mechanics behind concrete steps and specifically opening knee joints. The importance of development lengths and the location of resultant compressive forces within the step have been shown to be the decisive factors in the final effective operation of such.

In the design of concrete steps or opening knee joints, it is found to be critical to ensure adequate development length of tensile reinforcing into zones of compressive concrete. It is also noted that the width of vertical beam (or adjoining beam overlap) is equal or greater than the adjoining member’s depth and width of section, and that the size and depth of reinforcing in the section is retained throughout the entire mechanism.

CHAPTER 6

CONCLUSIONS AND IMPLICATIONS

6.1 Introduction

This report has outlined the profound importance of maintaining moment continuity throughout structural concrete beam steps. While a variety of different methods and arrangements are available in the attempt to achieve moment continuity, practical testing has uncovered the potential for large losses in flexural strength if this is done incorrectly.

6.2 Discussion

The satisfactory arrangement of step reinforcing has not been found to be new engineering principles, but old principles applied in new ways with a degree of engineered common sense. From stepped concrete beam testing it has been found that most deficiencies in design and performance were largely based on the lack of development length for reinforcing bars acting in tension. Without this effective length, the reinforcing bars do not acquire the ability to ‘develop’ maximum yield stress and therefore are prematurely displaced due a reduction in concrete to steel friction. This was the failure mode observed in 3 of the 5 test beam specimens.

Similarly, the testing produced results demonstrating poor efficiency based on the development length’s location for tensile reinforcing bars. Critical loss of strength was observed if such development was positioned in areas of concrete tension.

The choice of steel reinforcing arrangements within the step envelope for the 4 different beams proved to be successful, with the performance of each beam developing failure in dissimilar ways. Along with finding the performance of predicted unsatisfactory results, it also outlined the unexpected performance of a version of satisfactory performance based on the scaled dimensions of AS 2870. Ultimately it is these findings that have already influenced amendments to professional design details that had previously followed the arrangement of what was thought to be satisfactory. It is outcomes such as these, which have made the testing and research of this project worthwhile.

6.3 Further research and recommendations

Testing has uncovered the constraints in the AS 2870 for its beam step detailing, in respect to the notation of bar development length requirements. This notation suggests the required tensile bar extension has two choices of length; however it does indicate the minimum length required from the two given. This is concluded to be a defective design, especially when based on full scaled foundations. It is also noted that the step height restrictions enforced by AS2870 of '2D' is not a required limit, based on the assumption that no additional loads (i.e. soil pressure) are present throughout the foundation step.

It is the recommendation based on the findings of this research, that the notation of AS 2870 Figure 5.6 be amended to reflect the importance of satisfactory development length to all tensile reinforcing steel. To conservatively simplify this requirement, a minimum length of 40 times the bar diameter (40Ø) is suggested as the correct amendment to the current notation of *"25 Bar Diameters"* and complete removal of *"300"*.

Along with the findings of this research on the maintenance of moment continuity in structural concrete beam steps (with the focus on building foundations), there is endless scope to expand and research similar performance characteristics on many other concrete beam foundation details and connections. It is hoped that this direction of research will continue in the future and understanding along with development of structural concrete beams will continue.

6.4 Conclusions: Chapter 6

This chapter has outlined that from design, construction and testing of stepped concrete beams, the placement of tensile reinforcing steel can greatly influence the flexural strength and deflection behaviour when compared back to a simply supported comparative beam. It has found that along with varying results, the design clauses and limitations listed in *AS 2870 – Residential Slabs and Footings* does not outline the requirements of stepped beams found from practical testing. It is hoped that this research will help professional engineers, concreter's and builders alike in understanding the prerequisites of the design to maintain the moment continuity of structural concrete beam steps.

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APPENDIX A

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

<u>ENG4111/4112 Research Project</u>
PROJECT SPECIFICATION

FOR:	**David WALDOCK (BENG – Civil)**
TOPIC:	<u>The Maintenance of “Moment Continuity” within Structural Concrete Beam Steps and points of Possible Discontinuity.</u>
SUPERVISOR:	Associate Professor Dr Thiru Aravinthan Mr Lindsay Reid, RPEQ 2014
SPONSER:	Reid Consulting Engineers Pty Ltd
PROJECT AIM:	This research project aims to investigate the methods of arranging reinforcing steel at critical joint locations in concrete structures, such as vertical steps and inclined steps to maintain moment capacities under imposed ground actions.

PROGRAMME: **(Issue A, 15TH March, 2008)**

7. Research the fundamental engineering principles on structural concrete beams, and the ‘moment’ forces produced on critical joint locations.

8. Design scaled reinforced concrete beams of various step configurations used in common real world structural examples.

9. Construct and load to failure scale reinforced concrete beams, detailing all load, stress-strain and ultimate load/deflection characteristics to various step configurations.

10. Analyse and compare results of such testing with preliminary design calculations.

11. Evaluate and define optimum steel reinforcement configurations to maintain maximum moment continuity at structural concrete beam steps.

12. Outline the adverse effects of moment discontinuity in similar beam locations.

As time permits:

13. Expand the research and practical testing to other critical locations in structural concrete beam design.

APPENDIX B

BEAM TESTING DATA

Time Sec	Position mm	Load kN	Cycle
0	26.2951	0.011265	1
1	26.2353	-0.00703	1
2	26.2203	-0.00831	1
3	26.196	0.013292	1
4	26.1491	-0.04047	1
5	26.11	-0.05138	1
6	26.0866	-0.08237	1
7	26.0604	-0.07832	1
8	26.0237	-0.08279	1
9	25.982	-0.07302	1
10	25.9419	-0.11075	1
11	25.9126	-0.06446	1
12	25.8887	-0.05811	1
13	25.8647	-0.08422	1
14	25.8208	-0.05585	1
15	25.7726	-0.06714	1
16	25.742	-0.05698	1
17	25.7223	-0.04917	1
18	25.7	-0.04587	1
19	25.6573	-0.05245	1
20	25.6106	-0.0523	1
21	25.5722	-0.08884	1
22	25.5515	-0.08845	1
23	25.5276	-0.11006	1
24	25.497	-0.11367	1
25	25.4528	-0.09036	1
26	25.4105	-0.12347	1
27	25.3717	-0.11075	1
28	25.3479	-0.09415	1
36	25.0999	-0.0971	1
37	25.0551	-0.07305	1
38	25.0047	-0.10911	1
39	24.969	-0.12469	1
40	24.9541	-0.12884	1
41	24.9349	-0.11084	1
42	24.8909	-0.08026	1
43	24.8391	-0.11432	1
44	24.8023	-0.12654	1
45	24.7873	-0.15044	1
46	24.7664	-0.19991	1
47	24.7238	-0.26614	1
48	24.6717	-0.41515	1
49	24.6362	-0.5717	1
50	24.6267	-0.59125	1
51	24.6024	-0.70372	1
52	24.551	-0.97203	1
53	24.5023	-1.10194	1
54	24.4785	-1.23495	1
55	24.4649	-1.293	1
56	24.4335	-1.45957	1
57	24.3787	-1.70434	1
58	24.3382	-1.95453	1
59	24.3122	-2.09188	1
60	24.2949	-2.15638	1
61	24.271	-2.29147	1
62	24.218	-2.57793	1
63	24.168	-2.82559	1
64	24.1408	-2.97007	1
65	24.1266	-3.00851	1
73	23.8328	-3.80644	1
74	23.811	-3.76502	1
75	23.798	-3.78606	1
76	23.7687	-3.85436	1
77	23.7162	-3.95063	1
78	23.6711	-4.00978	1
79	23.6402	-3.99342	1
80	23.6258	-3.98105	1
81	23.6049	-4.01205	1
82	23.5596	-4.12786	1
83	23.5049	-4.20269	1
84	23.4645	-4.23157	1
85	23.458	-4.19593	1
86	23.4403	-4.21917	1
87	23.3911	-4.36828	1
88	23.3331	-4.44701	1
89	23.3027	-4.45485	1
90	23.2967	-4.45691	1
91	23.2738	-4.49106	1
92	23.2135	-4.6519	1
93	23.165	-4.74939	1
94	23.1498	-4.74086	1
95	23.1341	-4.73067	1
96	23.1008	-4.82509	1
97	23.0426	-4.81644	1
98	22.9986	-4.84517	1
99	22.9893	-4.75204	1
100	22.9777	-4.75046	1
101	22.9199	-4.94468	1
102	22.8643	-5.07086	1

29	25.3268	-0.09558	1	66	24.1058	-3.13291	1	103	22.848	-5.01287	1
30	25.3015	-0.09796	1	67	24.0535	-3.40784	1	104	22.8732	-4.85766	1
31	25.2563	-0.1187	1	68	24.0028	-3.54871	1	105	22.8878	-4.77982	1
32	25.2104	-0.12222	1	69	23.9725	-3.56898	1	106	22.8705	-4.86562	1
33	25.169	-0.10264	1	70	23.9595	-3.55503	1	107	22.8484	-4.97076	1
34	25.1443	-0.10416	1	71	23.9387	-3.60525	1	108	22.8542	-4.87405	1
35	25.127	-0.10771	1	72	23.8831	-3.72446	1	109	22.877	-4.77347	1
110	22.8865	-4.69875	1	147	22.8618	-4.69419	1	184	22.1045	-5.52917	1
111	22.8671	-4.80297	1	148	22.8484	-4.74769	1	185	22.0963	-5.49561	1
112	22.8499	-4.91083	1	149	22.8461	-4.80038	1	186	22.0363	-5.6909	1
113	22.8545	-4.83963	1	150	22.8671	-4.62815	1	187	21.9729	-5.7708	1
114	22.8749	-4.73118	1	151	22.8838	-4.5462	1	188	21.9581	-5.77638	1
115	22.8844	-4.64833	1	152	22.8892	-4.52879	1	189	21.9544	-5.73984	1
116	22.8784	-4.722	1	153	22.8702	-4.66624	1	190	21.9168	-5.83616	1
117	22.8593	-4.81561	1	154	22.852	-4.72707	1	191	21.8463	-5.9832	1
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119	22.8591	-4.82732	1	156	22.8524	-4.73458	1	193	21.8119	-5.92375	1
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122	22.8786	-4.71354	1	159	22.8846	-4.57323	1	196	21.6743	-6.17689	1
123	22.8554	-4.807	1	160	22.8828	-4.54351	1	197	21.6598	-6.12292	1
124	22.8453	-4.8652	1	161	22.87	-4.65336	1	198	21.6571	-6.0986	1
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126	22.8778	-4.61966	1	163	22.7572	-5.11771	1	200	21.5555	-6.32659	1
127	22.8893	-4.60669	1	164	22.7574	-5.0759	1	201	21.5084	-6.34983	1
128	22.8763	-4.63709	1	165	22.7627	-5.02837	1	202	21.497	-6.33803	1
129	22.8551	-4.77836	1	166	22.7102	-5.26297	1	203	21.4902	-6.32924	1
130	22.8443	-4.83122	1	167	22.6477	-5.39428	1	204	21.4468	-6.42815	1
131	22.8564	-4.76724	1	168	22.616	-5.39115	1	205	21.3849	-6.53827	1
132	22.8785	-4.63697	1	169	22.6118	-5.36999	1	206	21.3409	-6.56778	1

b

133	22.8888	-4.56634	1	170	22.5941	-5.39783	1	207	21.335	-6.4885	1
134	22.8801	-4.61847	1	171	22.5358	-5.51367	1	208	21.327	-6.52239	1
135	22.8602	-4.75195	1	172	22.4863	-5.53235	1	209	21.2797	-6.61442	1
136	22.8468	-4.81048	1	173	22.4474	-5.54708	1	210	21.2198	-6.72907	1
137	22.847	-4.78816	1	174	22.4307	-5.55566	1	211	21.1732	-6.78024	1
138	22.8676	-4.6919	1	175	22.4151	-5.5398	1	212	21.1652	-6.72653	1
139	22.8825	-4.58619	1	176	22.3925	-5.58668	1	213	21.1578	-6.69512	1
140	22.8897	-4.53809	1	177	22.3346	-5.71096	1	214	21.1168	-6.82253	1
141	22.8685	-4.71491	1	178	22.2714	-5.75694	1	215	21.0509	-6.95187	1
142	22.8461	-4.81808	1	179	22.255	-5.43243	1	216	21.0063	-6.98528	1
143	22.8428	-4.81254	1	180	22.2509	-5.3753	1	217	21.0054	-6.91158	1
144	22.8683	-4.65089	1	181	22.223	-5.48208	1	218	20.9972	-6.8931	1
145	22.8864	-4.56002	1	182	22.1544	-5.60778	1	219	20.9386	-7.05123	1
146	22.8856	-4.55993	1	183	22.1067	-5.60516	1	220	20.8794	-7.13226	1
221	20.8434	-7.18477	1	258	19.6635	-8.18896	1	295	18.4029	-9.71499	1
222	20.8383	-7.11915	1	259	19.6125	-8.30018	1	296	18.3437	-9.85503	1
223	20.8282	-7.12797	1	260	19.549	-8.41558	1	297	18.3148	-9.81814	1
224	20.7793	-7.25672	1	261	19.5033	-8.47483	1	298	18.3139	-9.76521	1
225	20.7186	-7.29424	1	262	19.5016	-8.41325	1	299	18.2959	-9.79283	1
226	20.6705	-7.22784	1	263	19.4937	-8.38554	1	300	18.2344	-9.93136	1
227	20.6673	-7.14642	1	264	19.4496	-8.49768	1	301	18.1739	-10.0369	1
228	20.6632	-7.13664	1	265	19.3893	-8.61165	1	302	18.1627	-9.95451	1
229	20.6134	-7.28381	1	266	19.338	-8.6664	1	303	18.2056	-9.8258	1
230	20.5499	-7.41461	1	267	19.3248	-8.61701	1	304	18.2188	-9.73067	1
231	20.5042	-7.44235	1	268	19.3206	-8.60855	1	305	18.1809	-9.86165	1
232	20.5047	-7.36731	1	269	19.2934	-8.6579	1	306	18.1616	-9.91246	1
233	20.4971	-7.40254	1	270	19.2312	-8.77753	1	307	18.1854	-9.783	1
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235	20.3785	-7.52994	1	272	19.1491	-8.8152	1	309	18.2098	-9.6859	1
236	20.3404	-7.52485	1	273	19.1462	-8.79109	1	310	18.1786	-9.78407	1

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238	20.3329	-7.46986	1
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243	20.1636	-7.66545	1
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256	19.676	-8.26696	1
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332	18.1693	-9.70691	1
333	18.203	-9.57727	1
334	18.2184	-9.52697	1
335	18.1914	-9.62341	1
336	18.1656	-9.73058	1
337	18.1679	-9.72474	1
338	18.1988	-9.59369	1
339	18.2168	-9.50757	1
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275	19.0715	-8.98591	1
276	19.0084	-9.02641	1
277	18.9779	-9.08631	1
278	18.9759	-9.03115	1
279	18.968	-9.02709	1
280	18.9068	-9.16561	1
281	18.8421	-9.29099	1
282	18.81	-9.28381	1
283	18.8129	-9.22716	1
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285	18.7358	-9.39423	1
286	18.6714	-9.47678	1
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288	18.6487	-9.38457	1
289	18.6309	-9.397	1
290	18.5665	-9.53993	1
291	18.5085	-9.62463	1
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294	18.4662	-9.58577	1
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370	18.216	-9.45196	1
371	18.1879	-9.60115	1
372	18.1653	-9.67246	1
373	18.1681	-9.62961	1
374	18.1985	-9.51469	1
375	18.2163	-9.46701	1
376	18.2072	-9.50324	1
377	18.1784	-9.62615	1

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314	18.2116	-9.6516	1
315	18.1832	-9.77644	1
316	18.1633	-9.82028	1
317	18.18	-9.73913	1
318	18.2079	-9.6283	1
319	18.2172	-9.58398	1
320	18.1858	-9.73079	1
321	18.1632	-9.78494	1
322	18.1761	-9.71112	1
323	18.2071	-9.60228	1
324	18.218	-9.58365	1
325	18.1873	-9.70793	1
326	18.1632	-9.77084	1
327	18.1725	-9.71064	1
328	18.2044	-9.59152	1
329	18.2181	-9.56285	1
330	18.1914	-9.66412	1
331	18.1652	-9.75963	1
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347	18.1609	-9.75543	1
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350	18.2067	-9.56011	1
351	18.1774	-9.64931	1
352	18.1611	-9.71943	1
353	18.185	-9.6136	1
354	18.2153	-9.50643	1
355	18.209	-9.54542	1
356	18.1818	-9.62555	1
357	18.1604	-9.69481	1
358	18.185	-9.60338	1
359	18.2148	-9.47523	1
360	18.2093	-9.5188	1
361	18.1782	-9.65148	1
362	18.1606	-9.70092	1
363	18.1882	-9.57954	1
364	18.2168	-9.45839	1
365	18.2054	-9.5095	1
366	18.1767	-9.64168	1
367	18.16	-9.70223	1
368	18.1846	-9.5982	1
443	16.0945	-11.9783	1
444	16.0581	-12.0825	1
378	18.1599	-9.68152	1
379	18.1808	-9.58514	1
380	18.2075	-9.46939	1
381	18.1556	-9.68409	1
382	18.0766	-9.95749	1
383	18.0562	-9.95177	1
384	18.0569	-9.9417	1
385	18.028	-10.01	1
386	17.9577	-10.1813	1
387	17.9078	-10.2535	1
388	17.9056	-10.2245	1
389	17.8993	-10.2412	1
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405	17.3178	-10.9435	1
480	14.8359	-12.8744	1
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415	16.9777	-11.3498	1
416	16.9498	-11.334	1
417	16.9488	-11.2991	1
418	16.9344	-11.2926	1
419	16.8725	-11.4183	1
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431	16.4432	-11.8589	1
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433	16.4281	-11.7714	1
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445	15.9946	-12.2151	1	482	14.7537	-12.8283	1	519	13.5273	-13.0562	1
446	15.9377	-12.2516	1	483	14.7505	-12.7922	1	520	13.5231	-12.9959	1
447	15.9282	-12.1972	1	484	14.7372	-12.777	1	521	13.502	-13.0314	1
448	15.9242	-12.1575	1	485	14.6711	-12.9173	1	522	13.4324	-13.1306	1
449	15.896	-12.2293	1	486	14.6072	-12.9349	1	523	13.3673	-13.1533	1
450	15.8308	-12.3214	1	487	14.5827	-12.8822	1	524	13.3541	-13.0709	1
451	15.7723	-12.3883	1	488	14.5818	-12.8395	1	525	13.3529	-13.0292	1
452	15.7577	-12.342	1	489	14.5706	-12.8311	1	526	13.3378	-13.0175	1
453	15.7544	-12.2932	1	490	14.5108	-12.933	1	527	13.275	-13.1107	1
454	15.7351	-12.2991	1	491	14.4468	-12.9712	1	528	13.209	-13.1788	1
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457	15.5846	-12.4593	1	494	14.3984	-12.8654	1	531	13.1736	-13.0384	1
458	15.587	-12.419	1	495	14.3538	-12.9727	1	532	13.1091	-13.1689	1
459	15.5698	-12.4251	1	496	14.2897	-13.0119	1	533	13.0367	-13.2247	1
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461	15.4386	-12.6259	1	498	14.2317	-12.9108	1	535	13.017	-13.0678	1
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471	15.1032	-12.7624	1	508	13.8767	-13.05	1	545	12.6852	-13.1156	1
472	15.0897	-12.7159	1	509	13.876	-12.9515	1	546	12.6713	-13.0871	1
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475	14.9983	-12.8121	1	512	13.7489	-13.0872	1	549	12.5176	-13.1635	1
476	14.9355	-12.8243	1	513	13.7054	-13.0829	1	550	12.5204	-13.0999	1
477	14.9254	-12.79	1	514	13.7012	-13.0183	1	551	12.5053	-13.1202	1
478	14.9198	-12.7446	1	515	13.6967	-12.9483	1	552	12.4308	-13.2543	1
479	14.9006	-12.7706	1	516	13.659	-13.0194	1	553	12.3667	-13.2675	1
554	12.3593	-13.1645	1	591	10.3687	-13.4774	1	628	7.88248	-13.4627	1
555	12.3553	-13.1423	1	592	10.2787	-13.429	1	629	7.84897	-13.422	1
556	12.3336	-13.1598	1	593	10.2384	-13.3809	1	630	7.81389	-13.4475	1
557	12.2642	-13.2683	1	594	10.1998	-13.3816	1	631	7.7249	-13.5292	1
558	12.2046	-13.2681	1	595	10.1379	-13.4331	1	632	7.62847	-13.5251	1
559	12.189	-13.1981	1	596	10.0412	-13.4914	1	633	7.54853	-13.483	1
560	12.1862	-13.1594	1	597	9.94686	-13.4804	1	634	7.51235	-13.4507	1
561	12.1707	-13.1697	1	598	9.89943	-13.38	1	635	7.47472	-13.4318	1
562	12.104	-13.2954	1	599	9.8638	-13.3857	1	636	7.40033	-13.5071	1
563	12.0366	-13.3145	1	600	9.81203	-13.4427	1	637	7.29851	-13.5244	1
564	12.0194	-13.2301	1	601	9.71119	-13.4695	1	638	7.21178	-13.5127	1
565	12.0173	-13.1775	1	602	9.61238	-13.4927	1	639	7.17528	-13.4577	1
566	12.0045	-13.1643	1	603	9.56342	-13.4281	1	640	7.1393	-13.4493	1
567	11.9363	-13.293	1	604	9.52874	-13.4098	1	641	7.06876	-13.5256	1
568	11.8707	-13.3341	1	605	9.4785	-13.4282	1	642	6.96796	-13.5119	1
569	11.8444	-13.2754	1	606	9.3837	-13.504	1	643	6.87856	-13.5377	1
570	11.8176	-13.2442	1	607	9.28434	-13.4953	1	644	6.83595	-13.4083	1
571	11.7173	-13.3783	1	608	9.22356	-13.4635	1	645	6.80174	-13.4286	1
572	11.6128	-13.3904	1	609	9.18854	-13.3987	1	646	6.74113	-13.4835	1
573	11.5675	-13.304	1	610	9.14466	-13.3974	1	647	6.64097	-13.5581	1
574	11.5349	-13.2949	1	611	9.05699	-13.511	1	648	6.54825	-13.5157	1
575	11.4685	-13.3457	1	612	8.95323	-13.5182	1	649	6.49894	-13.4346	1
576	11.3658	-13.4162	1	613	8.88392	-13.4904	1	650	6.46361	-13.4274	1
577	11.2825	-13.3757	1	614	8.85352	-13.4329	1	651	6.41171	-13.4555	1
578	11.245	-13.334	1	615	8.81318	-13.439	1	652	6.3134	-13.5272	1

579	11.2049	-13.3254	1	616	8.72552	-13.5293	1	653	6.2165	-13.5196	1
580	11.1311	-13.407	1	617	8.62548	-13.5281	1	654	6.15967	-13.4379	1
581	11.0261	-13.4258	1	618	8.54537	-13.4889	1	655	6.12628	-13.4109	1
582	10.9501	-13.4114	1	619	8.51454	-13.4307	1	656	6.08153	-13.4039	1
583	10.9138	-13.3571	1	620	8.47684	-13.3953	1	657	5.98465	-13.5077	1
584	10.8732	-13.3689	1	621	8.39822	-13.4752	1	658	5.88683	-13.4695	1
585	10.7957	-13.4081	1	622	8.29442	-13.5051	1	659	5.82012	-13.4195	1
586	10.6978	-13.4298	1	623	8.21445	-13.4867	1	660	5.7895	-13.3846	1
587	10.6141	-13.4034	1	624	8.18076	-13.4328	1	661	5.74616	-13.3938	1
588	10.576	-13.3832	1	625	8.14501	-13.4458	1	662	5.65629	-13.4633	1
589	10.5376	-13.3466	1	626	8.0645	-13.5246	1	663	5.55654	-13.4713	1
590	10.4668	-13.4375	1	627	7.95938	-13.5237	1	664	5.48323	-13.3919	1
665	5.44807	-13.3583	1	702	2.97881	-12.7918	1	739	0.496361	-11.932	1
666	5.40914	-13.3456	1	703	2.94338	-12.7885	1	740	0.461143	-11.8974	1
667	5.32863	-13.4336	1	704	2.86778	-12.8504	1	741	0.417066	-11.8866	1
668	5.22931	-13.4116	1	705	2.76521	-12.8318	1	742	0.319692	-11.9101	1
669	5.14527	-13.389	1	706	2.67591	-12.7555	1	743	0.22012	-11.8542	1
670	5.10894	-13.2888	1	707	2.63779	-12.7054	1	744	0.147611	-11.8125	1
671	5.06947	-13.3127	1	708	2.60207	-12.711	1	745	0.1142	-11.7218	1
672	5.00617	-13.3239	1	709	2.54165	-12.7466	1	746	0.077301	-11.7324	1
673	4.9047	-13.373	1	710	2.44324	-12.7339	1	747	0.002486	-11.7911	1
674	4.81106	-13.316	1	711	2.34646	-12.6803	1	748	-0.09955	-11.7548	1
675	4.76417	-13.2549	1	712	2.29045	-12.6247	1	749	-0.19387	-11.6869	1
676	4.72864	-13.2466	1	713	2.25715	-12.5559	1	750	-0.23088	-11.6413	1
677	4.67936	-13.2697	1	714	2.21391	-12.563	1	751	-0.26585	-11.6121	1
678	4.58228	-13.3103	1	715	2.12293	-12.5818	1	752	-0.32072	-11.6085	1
679	4.48407	-13.2807	1	716	2.02192	-12.5675	1	753	-0.42234	-11.6068	1
680	4.42269	-13.2193	1	717	1.94849	-12.4994	1	754	-0.52083	-11.5319	1
681	4.38518	-13.1685	1	718	1.91563	-12.4479	1	755	-0.57492	-11.4874	1
682	4.34647	-13.154	1	719	1.87704	-12.4477	1	756	-0.61026	-11.4185	1

683	4.26182	-13.2514	1
684	4.16002	-13.1795	1
685	4.07876	-13.1453	1
686	4.0427	-13.1105	1
687	4.00472	-13.069	1
688	3.93721	-13.1396	1
689	3.83354	-13.1165	1
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691	3.70381	-13.0203	1
692	3.66569	-12.9963	1
693	3.61018	-13.0308	1
694	3.51267	-13.0365	1
695	3.41535	-13.0123	1
696	3.3584	-12.9283	1
697	3.32181	-12.9132	1
698	3.2775	-12.8852	1
699	3.19376	-12.9484	1
700	3.08999	-12.9125	1
701	3.01241	-12.8569	1
776	-1.99198	-10.8167	1
777	-2.02788	-10.7243	1
778	-2.06219	-10.7196	1
779	-2.12046	-10.7131	1
780	-2.22365	-10.688	1
781	-2.32368	-10.6536	1
782	-2.37494	-10.5478	1
783	-2.4066	-10.5241	1
784	-2.4484	-10.5113	1
785	-2.54567	-10.516	1
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720	1.7985	-12.4911	1
721	1.69715	-12.4621	1
722	1.61051	-12.4481	1
723	1.56917	-12.3169	1
724	1.53195	-12.3428	1
725	1.48093	-12.3274	1
726	1.38184	-12.3605	1
727	1.28219	-12.3188	1
728	1.21859	-12.2073	1
729	1.18768	-12.2173	1
730	1.14634	-12.1877	1
731	1.06426	-12.2265	1
732	0.962904	-12.2037	1
733	0.875106	-12.1419	1
734	0.839594	-12.0891	1
735	0.803572	-12.0335	1
736	0.744921	-12.0687	1
737	0.643453	-12.0438	1
738	0.541994	-12.0153	1
813	-4.48077	0.051975	1
814	-4.50221	0.052154	1
815	-4.53452	0.053465	1
816	-4.59615	0.018179	1
817	-4.68551	0.01502	1
818	-4.77705	-0.00843	1
819	-4.85367	0.006914	1
820	-4.89257	0.013053	1
821	-4.9261	-0.02101	1
822	-4.98767	-0.03743	1
823	-5.08752	-0.02015	1
757	-0.65117	-11.4627	1
758	-0.74118	-11.4533	1
759	-0.8432	-11.4133	1
760	-0.92039	-11.3381	1
761	-0.95425	-11.2712	1
762	-0.99528	-11.2743	1
763	-1.06069	-11.2934	1
764	-1.16358	-11.283	1
765	-1.25745	-11.2158	1
766	-1.30403	-11.1233	1
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804	-3.81776	-9.64045	1
805	-3.85568	-9.57778	1
806	-3.93151	-9.59414	1
807	-4.03275	-9.51865	1
808	-4.12878	-9.43893	1
809	-4.16662	-9.36207	1
810	-4.20092	-9.32133	1
811	-4.25207	-9.29639	1
812	-4.35388	-9.25326	1
824	-5.17112	-0.05481	1
825	-5.10031	-0.04044	1
826	-5.07598	-0.01836	1
827	-5.12131	-0.02348	1
828	-5.13836	-0.01964	1
829	-5.10025	-0.05573	1
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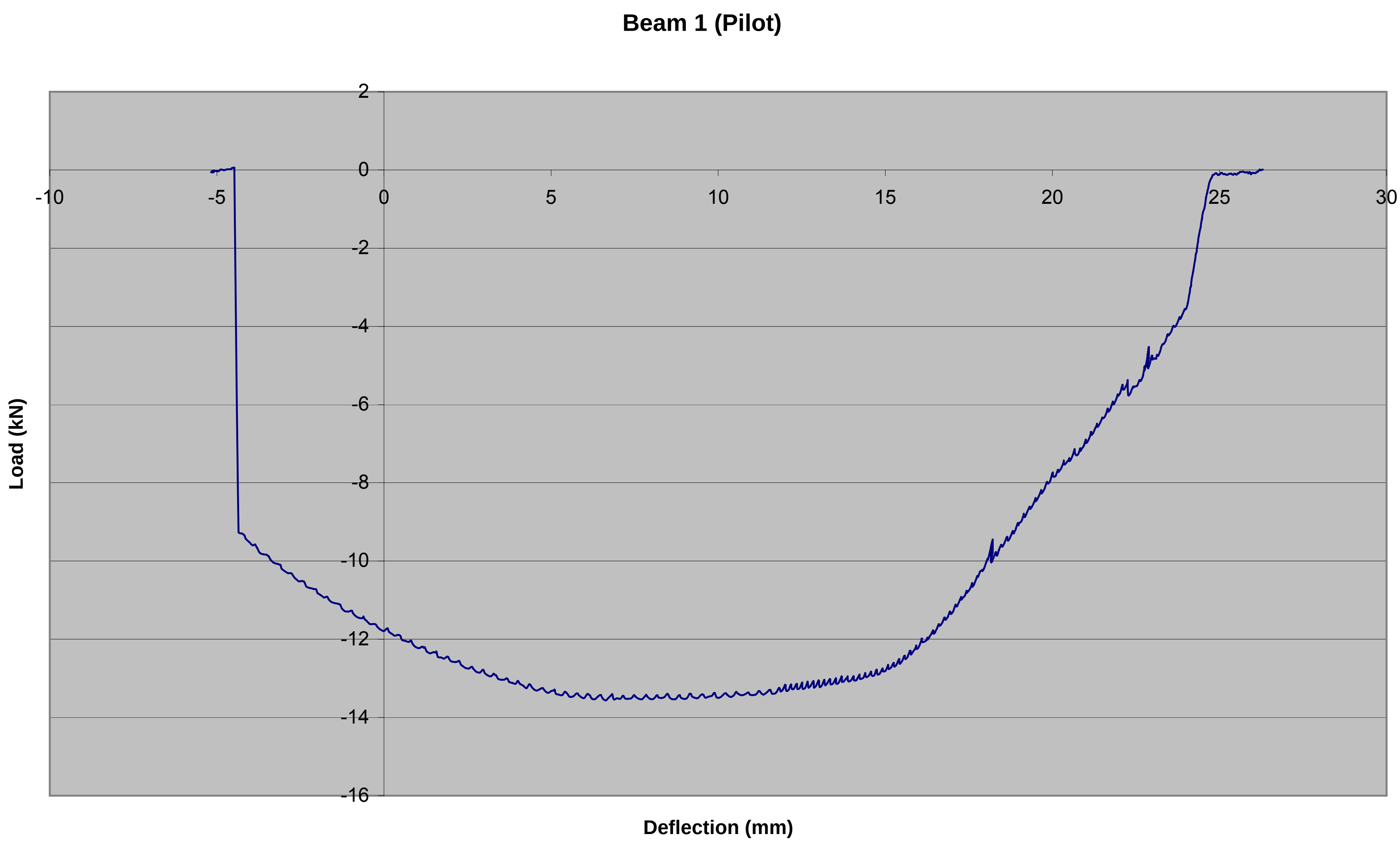
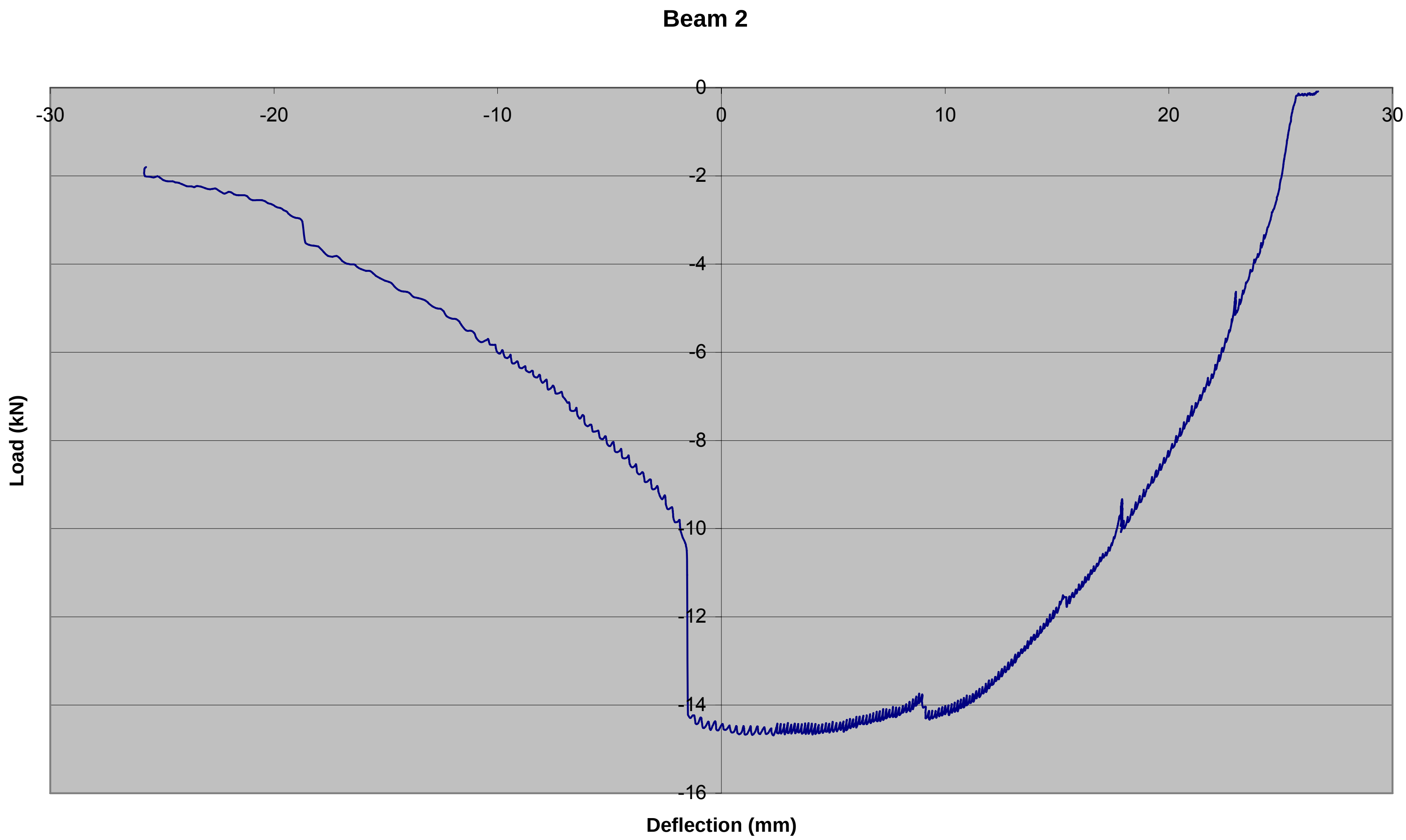
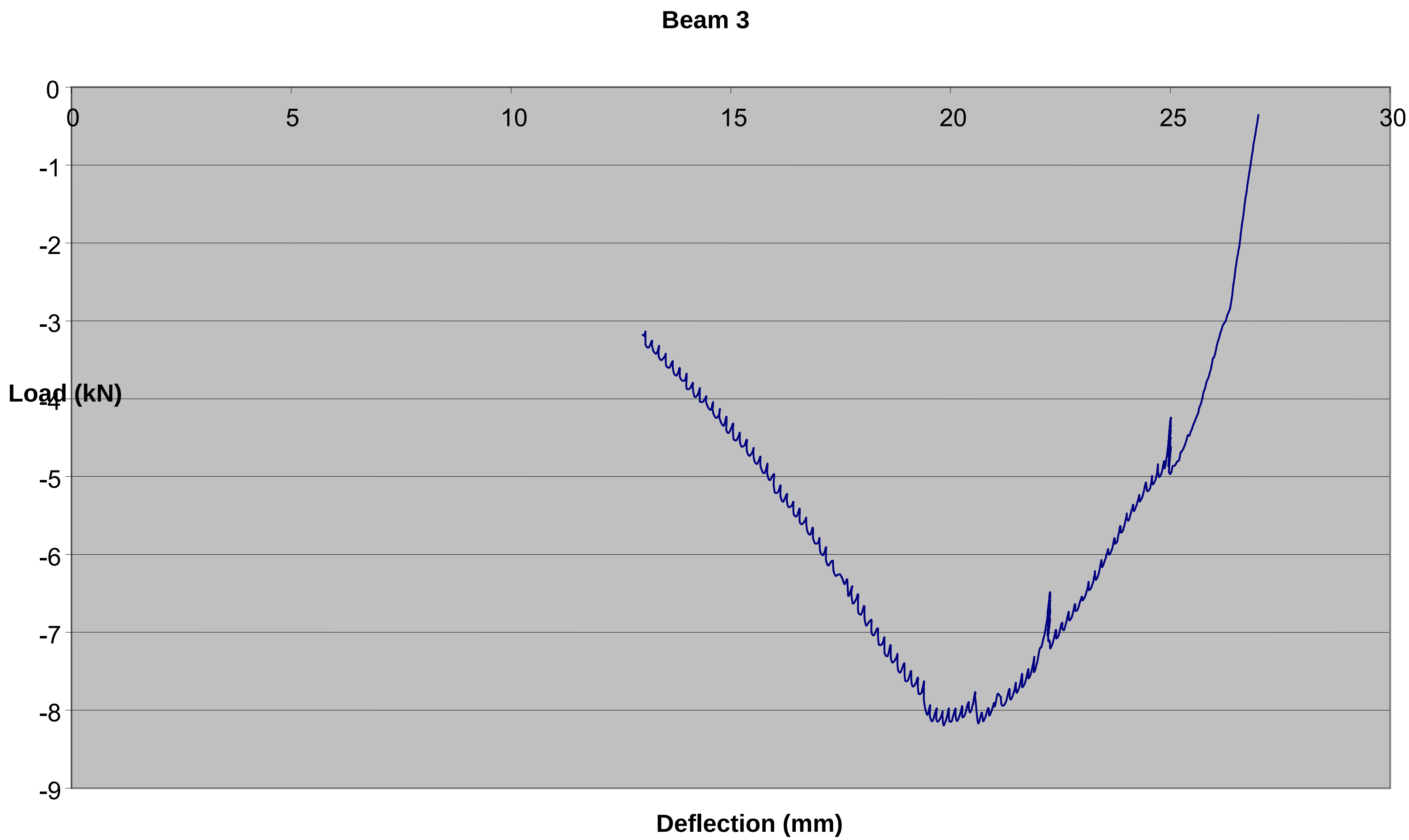

Beam 1 (Pilot)
Load (kN)
Deflection (mm)
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-10
-5
0
5
10
15
20
25
30

k



Beam 2
Load (kN)
Deflection (mm)
0
-2
-4
-6
-8
-10
-12
-14
-16
-30
-20
-10
0
10
20
30



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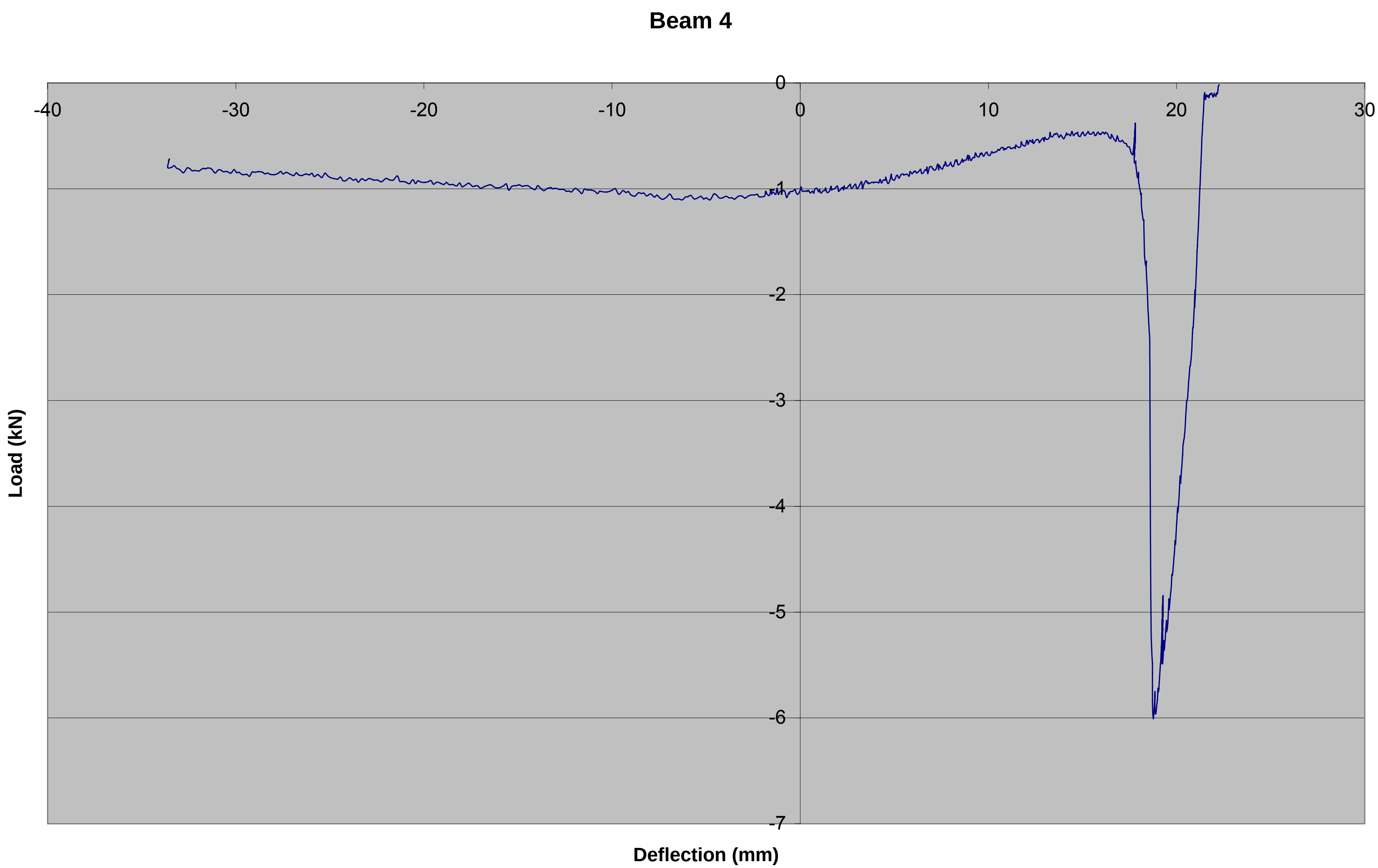

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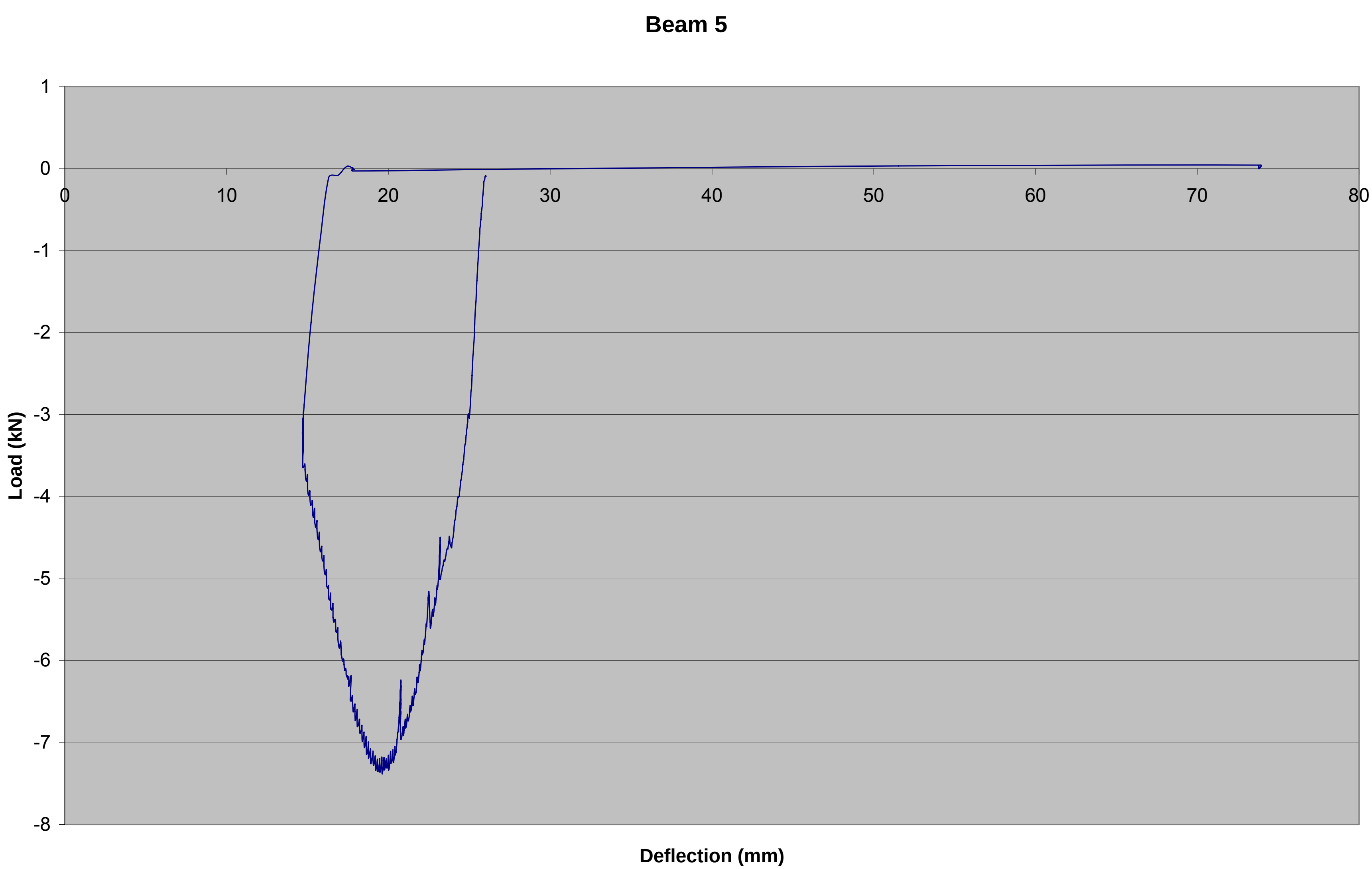
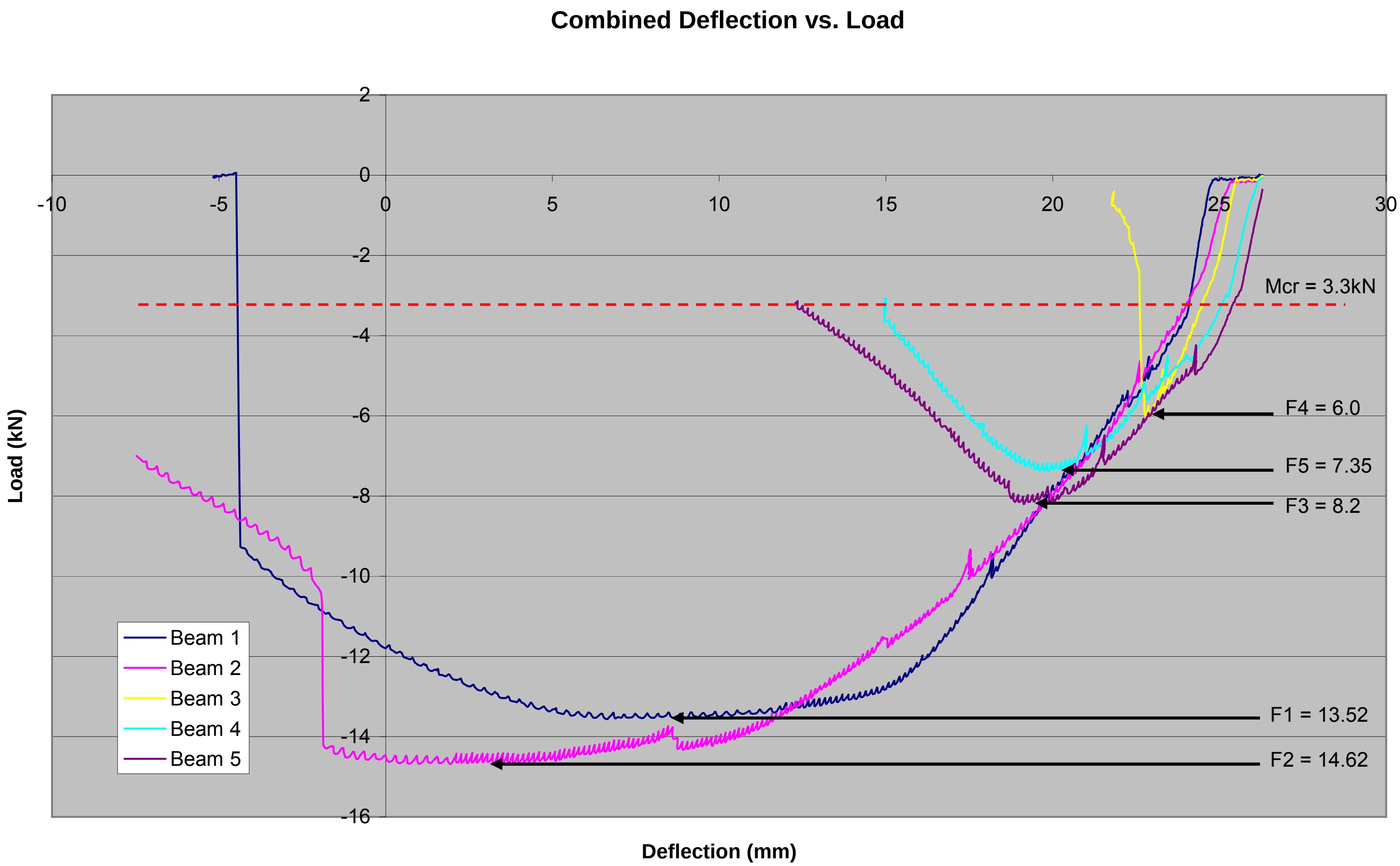

Beam 5
1
0
-1
-2
-3
-4
-5
-6
-7
-8
0
10
20
30
40
50
60
70
80
Load (kN)
Deflection (mm)

o


Combined Deflection vs. Load
Load (kN)
Deflection (mm)
2
0
-2
-4
-6
-8
-10
-12
-14
-16
-10
-5
0
5
10
15
20
25
30
Mcr = 3.3kN
F4 = 6.0
F5 = 7.35
F3 = 8.2
F1 = 13.52
F2 = 14.62
Beam 1
Beam 2
Beam 3
Beam 4
Beam 5

.

p