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SEISMIC RESPONSE OF CONNECTIONS IN PRECAST CONCRETE DOUBLE-TEES

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*In the Name of Allah
The Beneficent, The Merciful*

To

My Parents, My Wife, Fatemeh

and

My Brother Hadi's Spirit

Abstract

Thirty full-scale specimens of five types of connection were constructed and tested under: (a) monotonically increasing shear forces, (b) reversed cyclic shearing forces, (c) reversed cyclic axial forces, and (d) various combination of reversed cyclic shearing and axial forces- all increased gradually in stages until failure using displacement control. These connections were reinforced with anchor bars welded to the steel angle at angles of 45° and 90° , and/or with 90° standard hooks at the ends, headed studs welded to a steel plate and some combinations thereof, representing the connections between precast double-tee flanges, or between a double-tee flange and lateral load resisting elements. One full-scale specimen (Connection Type Ap) was designed and subjected to reversed cyclic shearing loads to investigate the efficiency of utilizing the steel angle versus the steel plate.

The effect of the varying ratios of reversed cyclic axial and shear forces on the different behavioral aspects of the connection, especially the shear response, including the hysteretic responses, yielding and ultimate loads, mode of failure, energy dissipation, displacement ductility ratios, etc. are reported.

Based on the experimental results, the shearing force - axial force interaction diagrams and the associated equations and their normalized versions are derived for the different connection types subjected to combined cyclic loads. The design interaction curves and the associated equation for each connection type are also developed along with the simplified version as a useful tool for design of the connections subjected to combined seismic in-plane shearing and axial forces induced in joints during an earthquake.

On the basis of the resisting mechanism observed experimentally and the physical models describing the ultimate behaviour of the connections, equations are proposed for shear and axial strengths of the various connections under static loads.

Non-linear finite element analyses, using the NONLACS program, are conducted to study the nonlinear response of the connections subjected to pure monotonically increasing pull-out and shear loads, and their combinations until failure. The analytical

interaction curves and the associated equations under combined monotonic pull-out (axial force) and shear forces are derived. The reduction factors to account for the lower strength under cyclic loads are derived empirically for these analytical equations. The simple modified analytical curves (for response under reversed cyclic loads) and their associated equations are developed for combined cyclic loading.

The experimental results were compared with the predictions made using proposed methods, the CPCI design method and non-linear finite element analysis.

In the final stage of this experimental-analytical program, practice-oriented methods are developed to determine the connection strength under shear and axial forces, and combinations of these forces at a joint between elements in a precast system. Design guidelines are provided for increasing the strength and the displacement ductility of the connections and to improve connection performance. It is concluded that properly designed and detailed connections of these types, except Connection Type E, can be designed to perform adequately in an earthquake-resistant structure.

Sommaire

Cette thèse présente une étude du comportement sous charges axiales et de cisaillement cycliques et de la résistance ultime de connexions utilisées entre les semelles des poutres en béton armé préfabriquées en forme de double T, ou entre une de ces semelles et des éléments de résistance aux forces latérales. Un total de trente spécimens pleine grandeur avec cinq types de connexions ont été construits et mis à l'essai sous les charges suivantes: (a) forces de cisaillement monotoniques, (b) forces de cisaillement cycliques inversées, (c) forces axiales cycliques inversées, et (d) diverses combinaisons de charges de cisaillement cycliques inversées et de charges axiales. Tous les essais sont du type destructif à déplacements (déformations) contrôlés. Ces connexions ont été renforcées avec barres d'ancrage soudées à la cornière en acier à des angles de 45° et 90° , avec des crochets standards à 90° aux extrémités, avec des gougeons d'ancrage soudés à une plaque d'acier, et des combinaisons des types énumérés. Un spécimen pleine grandeur (Type Ap) a été conçu et testé sous charges de cisaillement cycliques afin de comparer l'efficacité des connexions avec cornière et avec plaque en acier.

L'effet de la variation du rapport des charges axiales et des charges de cisaillement sur différents éléments du comportement de la connexion, en particulier la réponse en cisaillement et son caractère hystérétique, les charges d'écoulement et ultime, le mode de rupture, le degré de dissipation d'énergie, les rapports de ductilité, etc. a été étudié.

Des diagrammes d'interaction entre les forces axiales et les forces de cisaillement sont dérivés des résultats expérimentaux obtenus pour tous les types de connexions étudiés. Ces courbes d'interaction sont aussi obtenues sous forme d'équations, en version normalisée ou non. Une version simplifiée de ces courbes d'interaction constitue un outil de conception intéressant pour des connexions sollicitées par des forces axiales et de cisaillement dues aux séismes.

Les observations expérimentales ont également permis de proposer des valeurs de résistance axiale et en cisaillement pour des connexions sous charges statiques.

Des analyses non linéaires utilisant la méthode des éléments finis (logiciel NONLACS) ont été faites pour simuler la réponse non linéaire jusqu'à la rupture des connexions soumises à des charges de cisaillement et d'arrachement monotoniques, pures ou combinées. Des courbes d'interaction et les équations correspondantes ont été dérivées à partir des résultats de ces simulations. Des facteurs de réduction de résistance qui tiennent compte de l'effet des charges cycliques ont été dérivés empiriquement de ces courbes d'interaction analytiques. Ces facteurs de réduction permettent d'adapter les courbes d'interaction obtenues avec simulations de charges monotoniques au calcul de la réponse sous charges cycliques.

Les résultats des essais ont été comparés aux prédictions selon la méthode proposée (avec facteur de réduction pour charges cycliques), la méthode de conception du CPCI et l'analyse non linéaire par éléments finis.

A l'étape finale de cette étude, des méthodes pratiques simplifiées sont proposées pour déterminer la résistance d'une connexion entre éléments préfabriqués, soumise à des forces axiales et de cisaillement, seules ou combinées. Des recommandations sont faites pour augmenter la résistance et la ductilité des connexions en vue d'améliorer leur comportement global. Finalement, à l'exception de la connexion de type E, toutes les connexions étudiées peuvent être conçues et détaillées pour résister à l'effet des séismes.

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List of Symbols

A	effective stress area, or projected bearing area of the anchor
A_{cv}	area of concrete section resisting shear transfer
A_h	total area of anchor
A_s	area of reinforcement
A_{sh}	area of steel mesh
A_{vf}	area of shear-friction reinforcing steel
B_ϕ	factored bearing resistance
b_n	net thickness of concrete panel
C	average concrete cover thickness
C_1	cover thickness to the edge
C_r	factored resistance in compression reinforcing steel
C_u	ultimate compression strength in compression reinforcing steel
D_n	plan dimension of building in direction of computed eccentricity
d_b	diameter of steel bar or headed stud
d_{max}	measured crack width
e	distance between centre of mass and centre of rigidity
E_0	initial tangent modulus of elasticity of concrete
E_{sc}	secant modulus of elasticity of concrete at peak stress
E_c	tangent modulus of elasticity of concrete
E_s	modulus of elasticity of steel
E'_s	strain-hardening modulus
F	foundation factor
F_b	bearing strength against the bar anchor
F_h	bearing strength against the hook part anchor
f_c	measured concrete compressive strength
f'_t	splitting tensile strength of concrete
f_{ult}	measured ultimate strength of reinforcing steel
f_y	measured yield strength of reinforcing steel bar or stud anchor
f_{yh}	measured yield strength of steel mesh
I	importance factor of structure
k_1	modification factor to account for clear concrete cover thickness
k_2	modification factor to account for development length
l_d	anchorage length
l_{dbpo}	development length to prevent pull-out failure
l_{dbsp}	development length to prevent splitting failure
l_e	embedded length for studs or the effective length of the compressive leg
l_h	length of the right angle hook part
l_{hb}	basic anchorage length in hook anchorage

l_{hd}	straight lead of hook anchorage
n	number of bars in one layer of reinforcement or number of connections
P	experimental applied load
P_ϕ	factored tensile capacity of a connection
P_c	tensile strength of connection (concrete failure)
P_s	tensile strength of connection (steel failure)
P_u	factored tension loads
P_{ult}	ultimate static design load of connection
R	force modification factor
R_{ba}	inside bend radius
S	seismic response factor
s	spacing between connections in double-tee flanges
T	applied axial load or fundamental period of vibration of the structure
T_o	maximum force that connection can take in pure axial loading
T_r	factored resistance of tensile reinforcing steel
T_u	ultimate tensile strength of tension reinforcing steel
u, w	translational degrees of freedom
U	calibration factor
v	zone velocity ratio
V	applied shear load, or minimum lateral seismic force
V_c	shear strength of connection (concrete failure)
V_{ch}	shear strength of concrete surrounding the connection anchors
V_{dh}	strength of diagonal shear anchor crossing diagonal crack
V_e	equivalent elastic base shear
T_{cv}	contribution of the concrete in pull-out strength of the connection
$T_{c, ult}$	concrete cone-type pull-out strength of the connection
$T_{s, ult}$	pull-out strength of the connection govern by studs anchor
T_{hook}	contribution of each hook in pull-out strength of the connection
T_o	maximum force that connection can take in pure shear loading
T_{sv}	contribution of steel mesh in pull-out strength of the connection
T_{ult}	connection ultimate tensile strength due to pull-out of anchor studs
T_ϕ	factored axial force induced in connection during an earthquake
V_r	design shear force for connection
V_s	shear strength of connection (steel failure)
V_{sh}	contribution of steel meas in shear strength of connection
V_u	factored shear loads
V_{ult}	ultimate shear strength of connection
V_ϕ	factored shear force induced in connection during an earthquake
W	dead load of structure
Z_a	acceleration-related seismic zone
Z_v	velocity-related seismic zone
β	variable depending on type of connection
ϵ	strain

ϵ'_c	concrete strain at stress f'_c
ϵ_{cu}	strain at peak stress
ϵ_t	concrete strain at stress f'_t
ϵ_{ult}	ultimate strain of reinforcing steel
ϵ_y	yield strain of reinforcing steel
Δ_{ult}	measured displacement of connection at ultimate load
Δ_y	measured displacement of connection at yielding of reinforcing steel
λ	coefficient for use with low density concrete
μ	displacement ductility ratio or shear-friction coefficient
ρ	ratio of reinforcement
ϕ_c	resistance factor for concrete
ϕ_m	member resistance factor
ϕ_s	resistance factor for reinforcing steel
$\theta_z, \theta_x, \theta_y$	rotational degrees of freedom
ω	rotational normal to plan

Chapter 1

Introduction

1.1 Introduction

Over recent years, a large number of precast concrete structures have been built all over the world. Large panel construction is used mainly for residential buildings, while prefabricated structures made principally of linear elements are used for one to three storey industrial buildings.

The connections between precast members normally constitute the weakest link in the structure, and the satisfactory performance and economy of precast concrete structures depends to a great extent on the proper selection and design of the connections. The design and execution of these connections is therefore of the utmost importance and must be accomplished in a safe, rational and economical manner. Test results on full scale structures have shown that the connections start yielding even when steel in the precast panels is in the elastic range. This is because the strength of a connection is much less than that of the surrounding panel, and this strength can decrease with cyclic loading. The philosophy for the design of connections is:

- to remain elastic for small earthquakes
- to be able to undergo inelastic deformations without structural collapse.

Connections can be used to dissipate energy if they show stable elasto-plastic behaviour.

Some of the floor and roof slabs in early projects consisted of double-tees, single-tees, solid flat slabs, and many variations of these shapes. However, double tees have proved to be the most economical for spans of 9 to 21 m and have become standardized for industrial applications. Figure 1.1 indicates the components of a single - storey precast concrete wall panel industrial building. In precast building structures, seismic actions are assumed to be transmitted to the cast-in-place structural wall and/or frames through the roof and floors acting as horizontal diaphragms, which are assumed to act as

deep beams (CPCI, 1987). The lateral forces are transmitted from the diaphragm to the lateral load resisting elements as shearing forces in a deep beam. Therefore, it is essential to ensure adequate connections between the elements forming the roof diaphragm (usually double - tees with thin flanges) and between these elements and the lateral load resisting systems. During earthquakes, horizontal connections in the floor system are subjected to reversed in-plane cyclic shear and direct axial (tensile-compressive) loads. Figure 1.2 shows typical reversed cyclic forces induced in the connections due to the seismic loading. Therefore, the behaviour of a precast building during an earthquake is significantly affected by the strength and ductility of these connections subjected to the combined cyclic in-plane shear and axial forces. For the connections in the vicinity of the stiff lateral load resistant elements, significant axial forces are induced in addition to the shear forces (Kunze, 1965, Rasouli, 1991), therefore the current design philosophy should be modified to account for these forces. Due to the lack of sufficient data on the in-plane shear and axial resistance of connections between prefabricated elements, the designers make assumptions concerning the failure criteria, which may lead to unsafe or uneconomical designs. The design of these connections based solely on their static strengths is thus inappropriate. Some recorded data (Kunze, 1965) from the response of prefabricated structures subjected to earthquakes show that the most common failure mode occurs at the connection of the roof diaphragm with the boundary elements, which consist of reinforcing bars embedded in the thin flange by pull-out of the bar from one of the flanges. This emphasizes the importance of adequate embedment length and bond strength at the concrete-steel anchor interface for proper connection behaviour. Another common area of distress occurs at the connection of exterior precast panels (wall panels) to the diaphragm system. The welded embedded anchors exhibit little ductility, and failure often occurs in a brittle manner by pull-out or fracture of the weld between the reinforcing bar and the steel plate.

There is a considerable lack of experimental and analytical data on the resistance of such connections subjected to dynamic shear and normal forces and presently there is no published data for response of these connections to the combined forces induced by the earthquake. Also, very little research has been conducted to study the connection

response under monotonically increasing loads. The objective of this research program is to study the response of the connections between the double tee flanges in the diaphragm system as well as between double tee flange and the lateral load resisting elements such as wall panels, when subjected to combined reversed cyclic loads which act on the connections during an earthquakes.

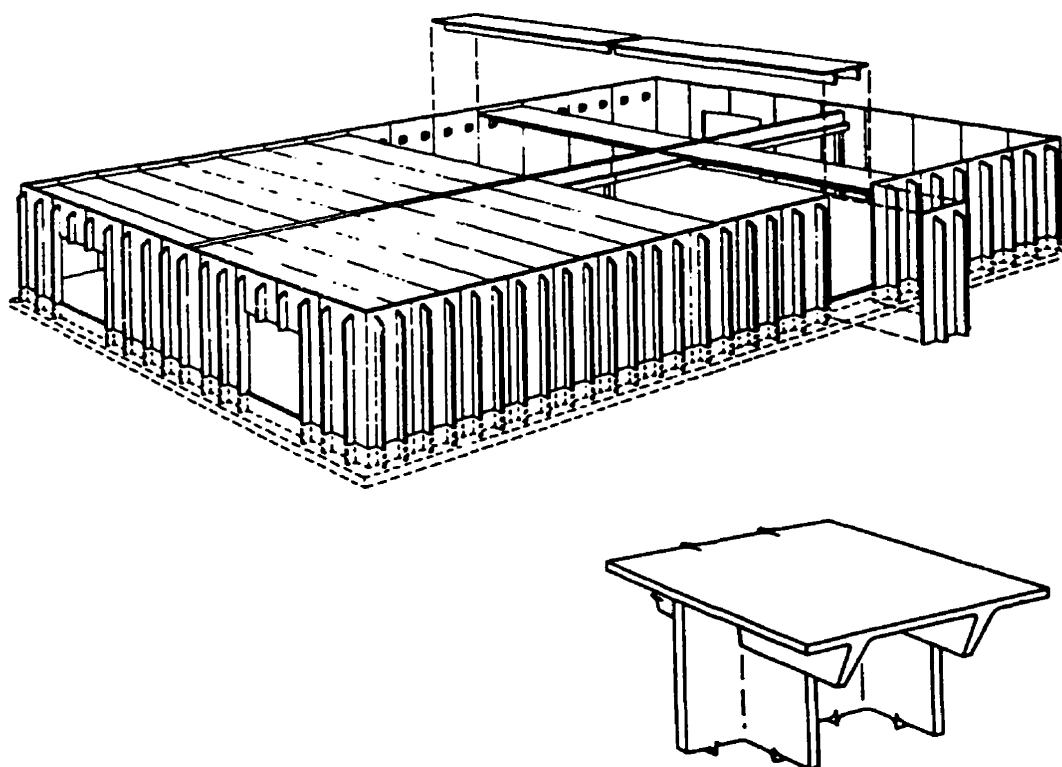


Figure 1.1 Framing system for a single-storey precast concrete wall panel building

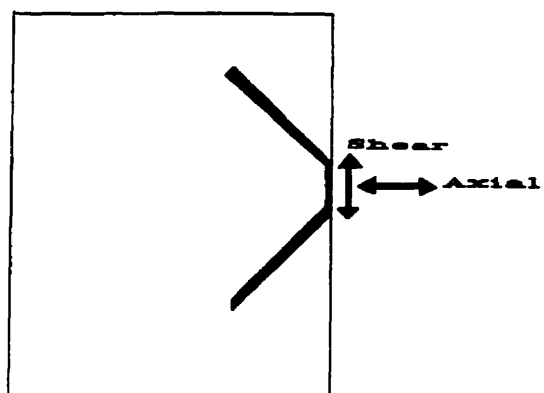
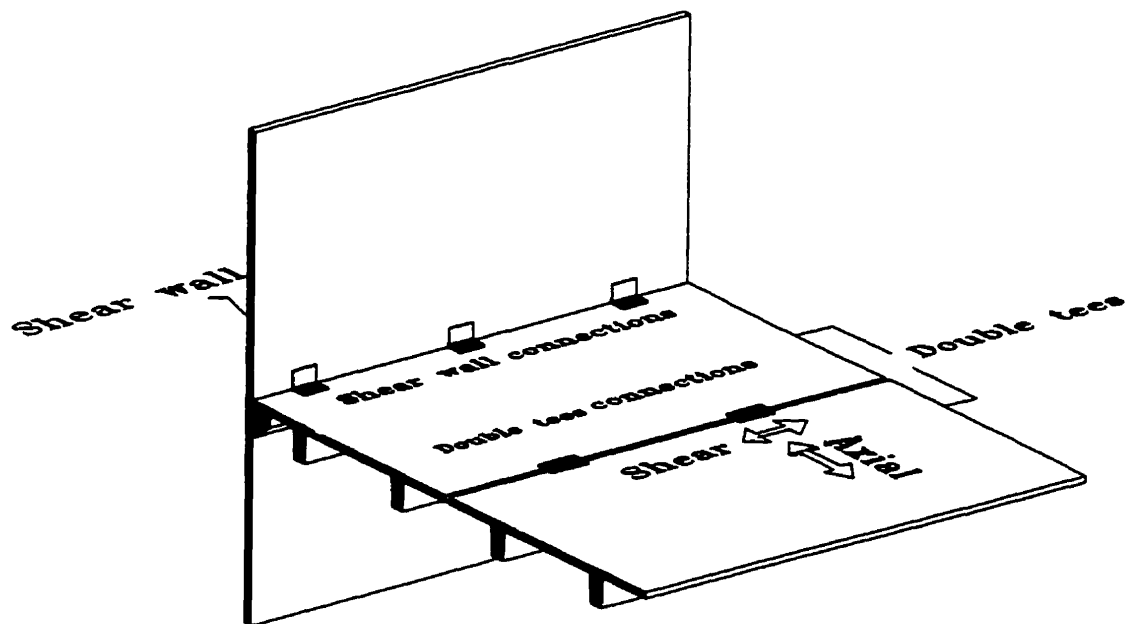


Figure 1.2 Typical connections and cyclic forces between double-tee elements

1.2 Summary of Previous Investigations

Previous studies on typical connections in precast concrete structures are summarized in this section

1.2.1 Design Concepts and Models

Analysis for the design of connections requires analytical models, which should be simple but should still reveal the essential features of the connection behaviour. Truss, or strut-and-tie models appear to be well suited, and they have been developed for most connections. They are useful because they allow a designer to identify the functions performed by each element of the connection and to obtain the approximate magnitude of the forces.

Hawkins (1977 and 1978) reviewed the seismic performance of precast concrete buildings, and noted the need for research in the following areas: determination of the design forces, design of elements, design of connections, contribution of floor systems, and provision of overall structural integrity.

Becker and Sheppard (1981) classified the types of connections by dividing the connection in precast concrete elements into three groups as follows:

- i) Strong connections - Under the ultimate seismic load, the connection does not exceed the elastic limit.
- ii) Ductile connections - The connection is designed based on the loading beyond the elastic range into the inelastic range to absorb energy, to achieve ductility, or to limit load transmission to the other elements in the structural system.
- iii) Guided connections - It permits displacement under the lateral loads (earthquake or wind) and adapts to the changes due to creep, shrinkage and temperature variations.

Stanton et al. (1986) suggested that due to the lack of special seismic design criteria and details, one of the following procedures may be used: (i) design the structure to remain in the elastic range; (ii) design the connection to be stronger than the precast member; (iii) design the structure so that the connections are located outside of potential

inelastic zones.

The models proposed for typical horizontal connections are summarized in the following:

Aswad (1979) used panels in combination as a horizontal joint which were modeled as an elasto-plastic model without any strength or stiffness degradation. Neille (1977) modelled the behaviour of welded headed stud connections by combining a trilinear load-deformation model with a stress - strain model for concrete adopted from the work of Karson and Jirsa (1969). Neille (1977) obtained very good agreement between the calculated behaviour of his analytical model and the measured cyclic behaviour of the experiment. Shricken and Powell (1980) used a variety of load - displacement relationships for the connection elements using a finite element approach. The shear friction element was developed for modelling the dry connection to capture the pinched hysteresis loops, stiffness at the end of each cycle and the degradation of stiffness and strength with an increasing number of load cycles. Shariatmadar and Mirza (1992 and 1995) modeled the reinforcing steel anchors using the embedded steel bar concept, while the concrete panels were idealized with quadrilateral shell elements with cubic displacement fields in both directions. They addressed the bond-slip characteristics at the interface of the concrete and steel bar. The evaluation of analytical results showed that the stiffness and strength of the connection decreased due to the cyclic loading. Shariatmadar (1992) proposed an alternative method to calculate the ultimate strength of the connections.

1.2.2 Experimental Investigations

In this section, a brief review of the experimental investigation of the response of some common connections between precast elements subjected to cyclic loads is presented. In spite of the concern and the recognized need for information on the cyclic loading response of precast connections, only a few tests have been conducted to study their behaviour. Again, there is very little data on the energy absorption and ductility characteristics of connections commonly used in precast construction. For convenience,

connections are classified into two general categories according to the material used: dry and wet connections. This report focusses on the description of the response of "dry" connections (using headed studs or embedded reinforcing bars).

Dry Connections

These connections utilize mechanical anchors such as welded headed studs and embedded reinforcing bars welded to a plate to transmit the shear forces between the precast elements such as double - tees and wall panels. The most common type of dry shear connections are composed of embedded steel shapes anchored into the precast members by studs or reinforcing bars. The connection is completed by bolting or welding a third element to the embedded steel shape. Figure 1.3 details some of the most commonly used dry connections.

Shear force is transferred across the connection by a variety of mechanisms. Depending on the details of the connection, shear is transferred through bearing of the steel shapes, shear of the connecting elements, shear of the welds or bolts, or through friction between the bolted plates. Many tests have been conducted to determine the ultimate strength of dry connections under monotonic increasing static loading, however, only a few tests have been carried out under reversed cyclic loads.

The research on cyclic behaviour of different types of welded connections is described briefly for: (i) headed stud connections, and (ii) embedded reinforcing bar connections.



Figure 1.3 Dry shear connectors

1.2.2.1 Connections with Headed Studs

A typical headed stud connection is presented in Figure 1.4 (Neille, 1977). The strength of the headed stud is limited by either the concrete or the stud failure. Neille and Spencer (1976) conducted tests on the behaviour of six welded headed stud connections subjected to cyclic loading. Each connection angle was cast in a 4 ft x 2 ft x 8 in (1219 mm x 610 mm x 203 mm) concrete panel. Details of the test connections are shown in Figure 1.5. Five of the test specimens were subjected to cyclic loading. Connection A1 was subjected to monotonically increasing load until failure. Reversed cyclic shear forces were applied to all other specimens at a distance of 7/8 inch (22 mm) from the face of the connection angle in the longitudinal direction (see Figure 1.5). Load-deflection curves for these connections are presented in Figure 1.6 (Spencer et al. , 1976). Test results for the connection A1 exhibited considerable ductility under monotonically increasing load (see Figure 1.6a). The yield strength of the connection was greater than the design strength predicted by the PCI equation (PCI, 1971 and 1985). The test results for Connections A subjected to reversed cyclic loads illustrate the change in the behaviour of the connection when subjected to either monotonically increasing or reversed cyclic loads. The connection was loaded to yield after several initial cycles in the elastic range, and then subjected to steadily increasing controlled deflections. The results show a decrease in stiffness and strength with load cycling. The failure of the connection was preceded by progressive crushing and spalling of the concrete above and below the angle. They also observed progressive tension cracking parallel to, but some distance away from the angle. Connections A1 and B1 failed when a large block of concrete surrounding the studs broke away while the other connections failed when one of the studs broke close to the weld. Spencer and Neille (1977) concluded as follows:

1. The PCI procedures for calculating the ultimate design strength of these connections under static loading give conservative results.
2. The strength of the connections in the first cycles of loading up to yield is approximately the same as the strength under monotonically increasing loads.
3. If the cyclic loading is continued above the stability limit [see Figure 1.6(b) for

definition], the strength of the connections decrease with an increasing number of cycles.

4. The deflections reached before failure were from seven to twenty-four times the theoretical elastic deflection corresponding to the design ultimate strength.

5. These connections, if properly designed and detailed, appear to be suitable for use in earthquake-resistant buildings designed as box - type systems.

Shariatmadar and Mirza (1992 and 1995) studied the behaviour of connections with welded headed studs in thin concrete panel (75 mm) subjected to reversed cyclic shear. The connection details and hysteresis response for the connection are presented in Figure 1.7. The experimental results showed narrow load-deflection loops indicating small energy dissipation. The ultimate strength of the connection was about 65% of the ultimate capacity under static loads, calculated using the CPCI method (CPCI, 1987). The failure mode was "shear-cone" type failure. They suggested using longer headed studs welded to the steel plate with longer distance between the studs thereby increasing the effective stress area which results in a higher strength of the connection.

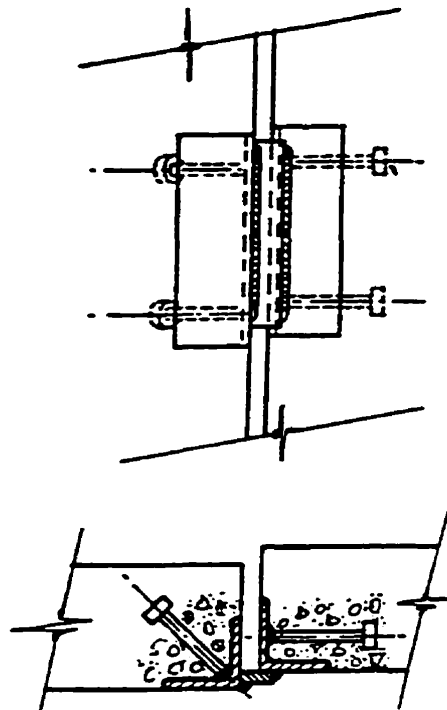
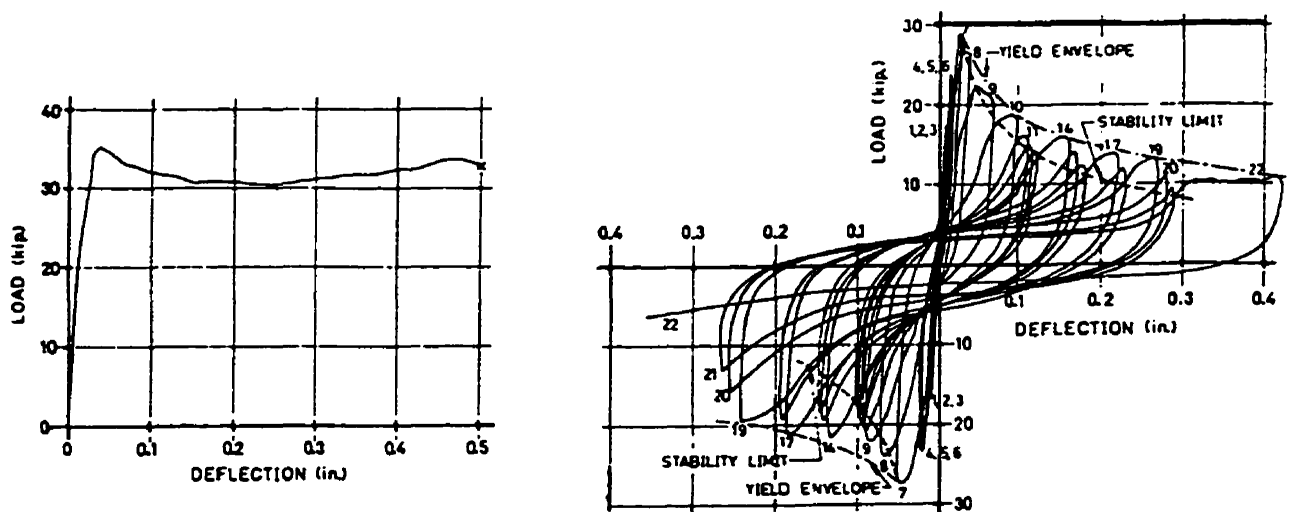


Figure 1.4 Typical headed studs connection demonstrating misalignment (Neille, 1997)

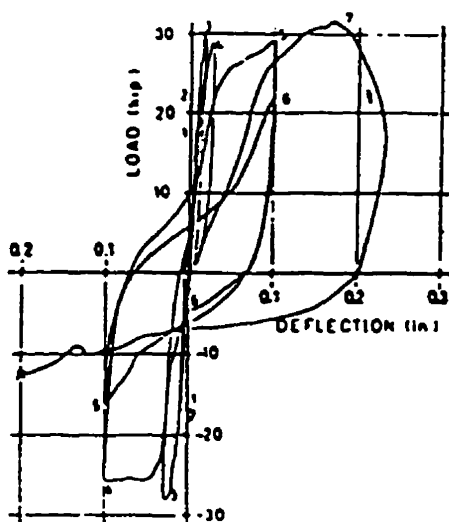
CONNECTION	DETAILS OF STUDS & PANEL REINFORCEMENT	
A1		
A2, A3		
B1		
B2		
B3		

Figure 1.5 Details of test connections (Spencer and Neille, 1976)

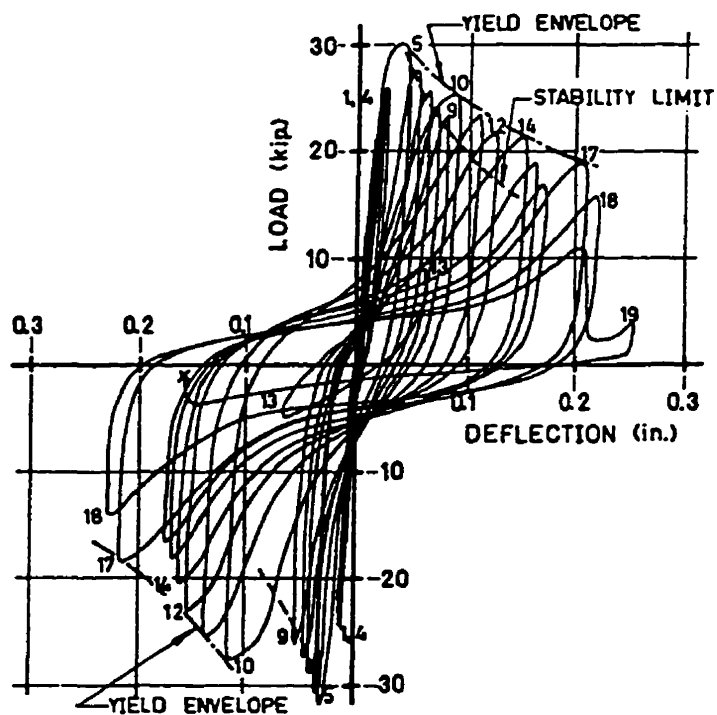


(a) Load-deflection curve (connection A1) (b) Load-deflection loops (connection A3)

Figure 1.6 Load-deflection curves for connections in Figure 1.5 (Spencer et al, 1976)

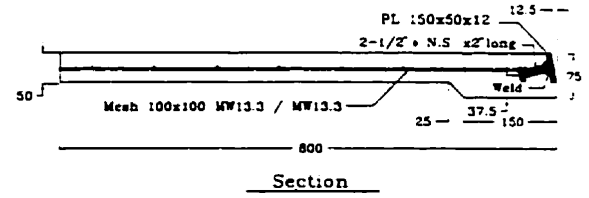
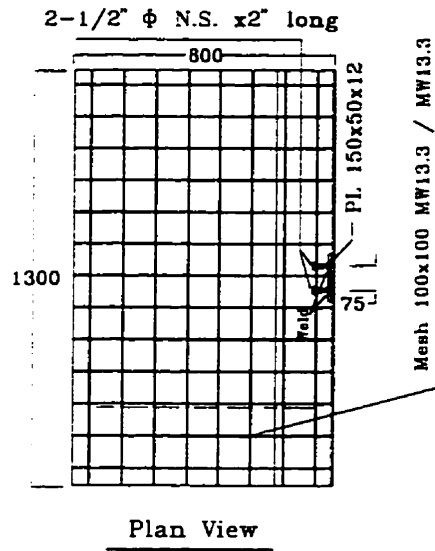


(c) Load-deflection loops (connection B1)

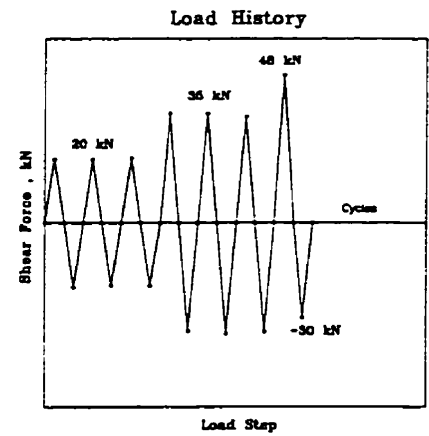
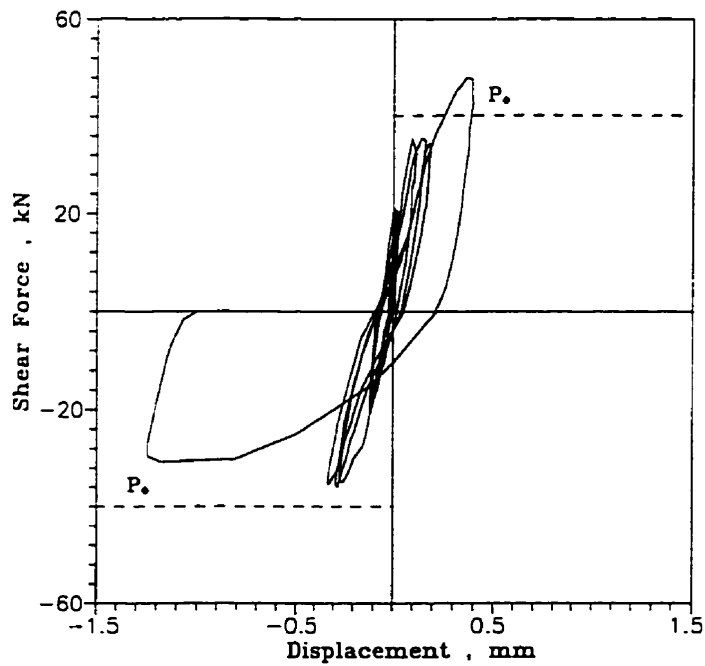


(d) Load-deflection loops (Connection B2)

Figure 1.6 (Continued) Load-deflection curve (Spencer et al, 1976)



(a) Details of the Specimen S4



(b) Loading history and hysteresis response

Figure 1.7 Specimen details and load-deflection curve for Connection S4 tested by Shariatmadar et al (1992)

1.2.2.2 Connections with Embedded Reinforcing Bar

An alternate method for embedding steel sections in precast members is to replace the headed studs with welded reinforcing bars (Figure 1.8), which are anchored in the concrete. Although sufficient test data is not available for connection responses under seismic loading, there are several advantages to this type of connection. The first advantage is the large surface area available for welding the reinforcing bars to the angle, which reduces the probability of brittle weld failure because of lower weld stresses. The second advantage is the larger development length of the bars into the precast member. This allows for lower bond stresses and reduces the probability of pull-out and splitting failure of the connection.

In general, dry connections should be designed conservatively for earthquake resistant construction. Longer weld lengths, additional anchorage lengths for reinforcing bars and lower allowable stresses are justified for earthquake loading especially in the presence of forces normal to the shear connection.

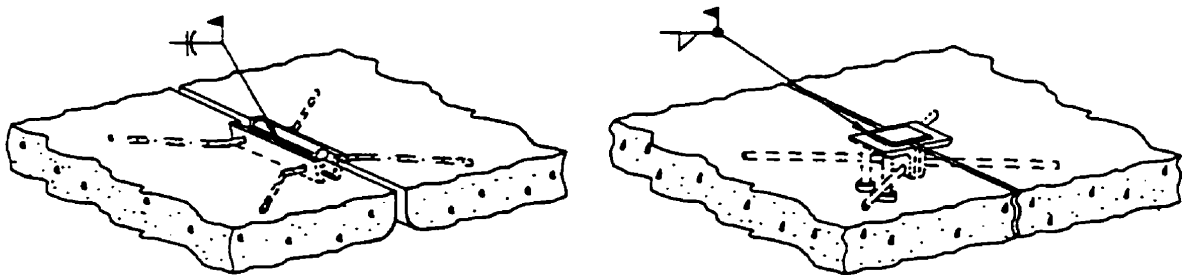


Figure 1.8 Connection with embedded reinforcing bar

A brief summary of the response of connections with anchored reinforcing bar subjected to reversed cyclic loads follows:

Aswad (1979) conducted a series of low cycle tests on connections using full scale double tees and wall panel (21' x 8' or 6.4 m x 2.4 m panels). Two types of connections for double tee (2" or 50 mm flange thickness) and for precast wall panels (6" or 150 mm thick) were tested. The experiment did not cover a wide range of cyclic tests because the main objective was to find out the failure strengths of the connections and to measure the relative displacements between the adjacent precast double tees. In a majority of

cases, the specimens were subjected to three cycles of reversed loadings between $\pm P_o$, where P_o is a value smaller than the ultimate strength and the connection was then brought to failure. However, the cyclic tests were not carried far enough to present the actual cyclic response of the connections. Details of the connections and the respective load-displacement curves are shown in Figure 1.9. The stiffness and strength did not deteriorate considerably since the reversed cyclic loads remained essentially in the elastic range. The following conclusions were reported by Aswad (1979)

1. When forces applied to the plate were simple shear, the failure strengths were greater than the strengths calculated using the imperial method.
2. The embedded length of the reinforcing bars were sufficient to develop the ultimate capacity of the anchor bar.

Spencer (1986) carried out some cyclic shear tests on connections with reinforcing bars anchored into typical 4'x 2'x 5.5" (1220 x 610 x 140 mm) concrete panels. The hysteresis response curves for the tests are presented in Figure 1.10. The following dynamic characteristics were observed by Spencer:

1. The strength of the connection does not reduce during loading cycles in the elastic range.
2. The nominal strength of the connections can be determined using the truss analogy.
3. The strength of the connection with the anchor bar running into the connection at 45 degrees and at 90 degrees falls to about 50% and less than 50% of their nominal strengths, respectively.
4. The connection with a recess in the panel edge and straight embedded bars at 45 degrees appears to perform best under simulated earthquake loading.
5. The panel thickness and concrete strength influence the behaviour of the connection.

Other tests on six different types of connections with embedded bar in thin panels were conducted by Kallros (1987). All specimens were loaded into the elastic range and then the loading was continued until the connection failed. The modes of failure occurred either by the bar fracture or by the spalling of the concrete. When bar failure occurred, the related measured deflection was very small. In the case of spalling failures, when the connections started cracking, the stiffness gradually decreased with an increasing number

of cycles into the inelastic range and larger load-deflection loops were observed. The details of the connections and the load - deflection curves are presented in Figure 1.11. Kallros (1987) stated the following conclusions:

1. Good quality control is necessary with the use of the reinforcing bar and the angle.
2. A connection with a thicker flange with a larger concrete cover thickness behaves better under cyclic loading.
3. The connections should be designed for cyclic load equal to 50% of the required capacity under monotonically increasing loads.
4. The strength of the connections does decrease in the elastic range.
5. Connections with 45 degree embedded bars appear to perform best under simulated earthquake loading.

Shariatmadar and Mirza (1996) conducted tests on full size connections with reinforcing bar embedded in thin panels, 50 mm thick which was enlarged to 75 mm in thickness at the edges. The specimens represent a series of connections between the double tees or between the double tee and the lateral load resisting element in a precast concrete structure. The connection types are shown in Figure 1.12 through 1.15. The connections S1, S2 and S5 were designed based on the analogy and the recent research results on bar development lengths under reversed cyclic loads from the University of Texas at Austin and Cornell University (ACI, 1989). The connection S3 was designed based on load transfer by the shear friction mechanism. The size and the boundary conditions for the experimental specimens were derived from linear finite element analysis of the double-tee with three connections along with the analysis of the test specimen with single connection. The connections were tested under reversed cyclic shearing forces, to investigate the effect of the different details and modes of failure on the overall response of the connections. Details of the connections and the respective loading history and hysteresis responses are shown in Figures 1.12 through 1.15. The envelopes for the load-deflection responses and the cumulative energy dissipated are compared in Figures 1.16 and 1.17, respectively. The Specimens S5 exhibited the largest ultimate strength, while the Specimen S2 exhibited the largest capability to dissipate energy. The Specimens S2 and S5 displayed almost similar load-deflection envelope pattern, although different

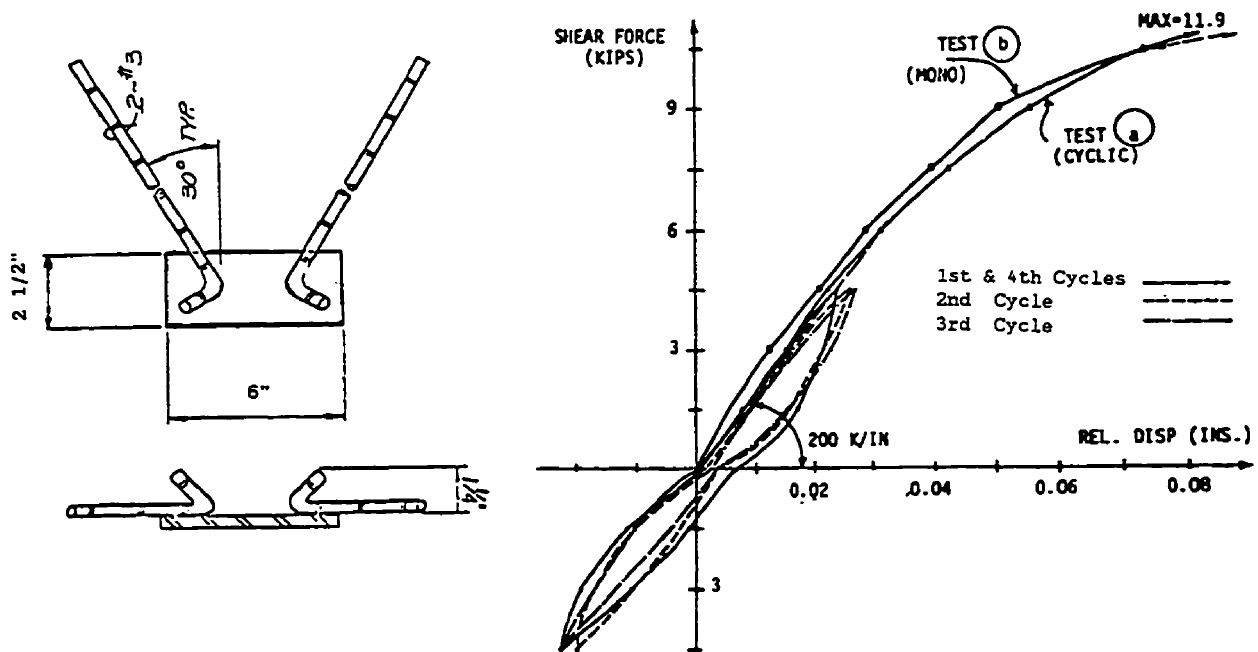
failure modes were observed. The failure modes are dominated by the concrete for all specimens, except for Specimen S1 which failed by fracture of the reinforcing bar close to the weld. Specimen S2 showed well-rounded and large responses due to the provision of the hook. It resulted in a considerable improvement in the energy dissipation ability, after the splitting bond failure occurred along the straight part of the anchor bar. In general, a more flexible envelope of the load-deflection response was an indicator of an increased stiffness degradation of the connection under reversed cyclic loading. They also noted that serious cracking of the anchorage zone was observed before noticeable yielding of the reinforcing anchor. In order to improve response, they suggested to use the development lengths of :

$$L_d = 154.5 d_b / \sqrt{f'_c} \geq 30d_b ,$$

$$\text{and } L_{hd} \geq 149.5 d_b / \sqrt{f'_c}$$

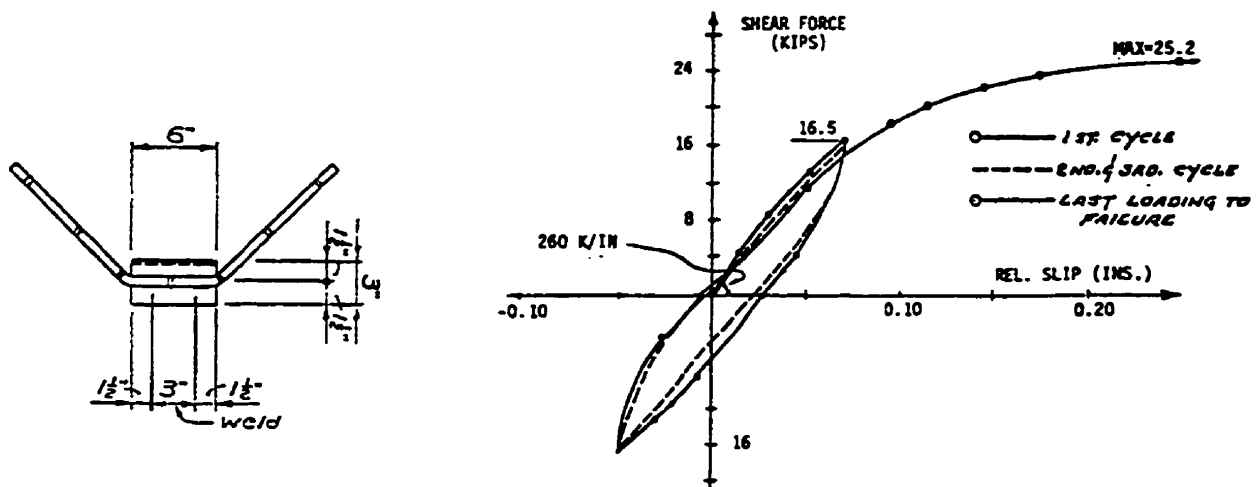
for straight and 90° hook anchor bar, respectively.

The guidelines were represented for designing and detailing such connections



(a) Shear force vs. relative slip for Specimen D-34

Figure 1.9 Connection Details and test results reported by Aswad (1976)



(b) Shear force vs. relative slip for Specimen P-9

Figure 1.9 (Continued) Connection details and test results reported by Aswad (1976)

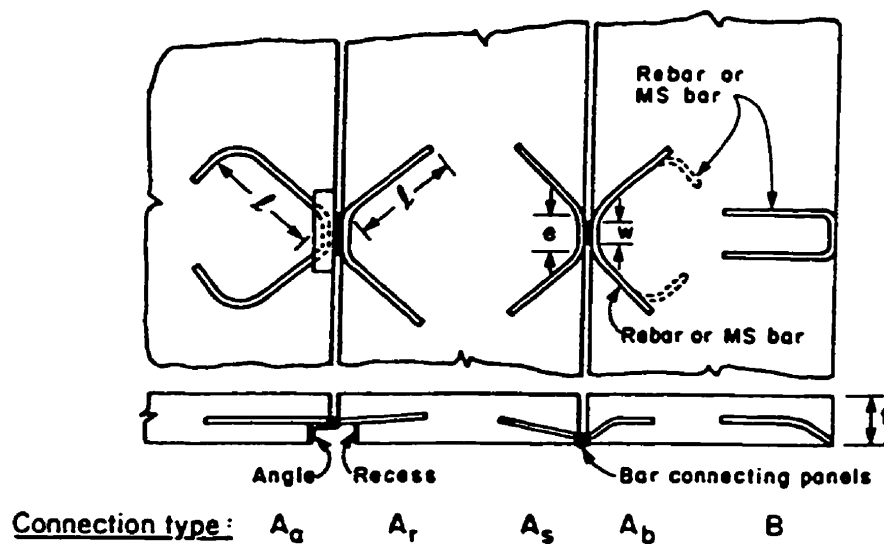
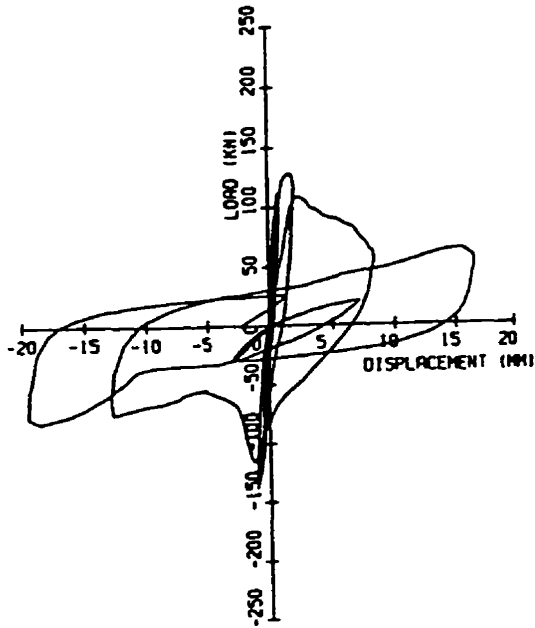
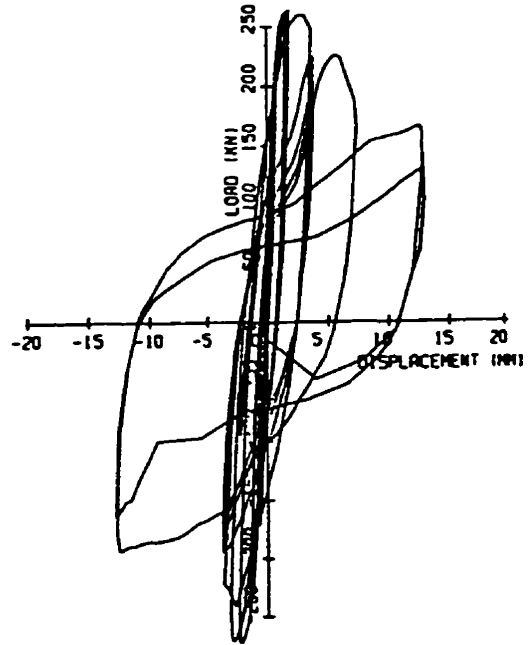


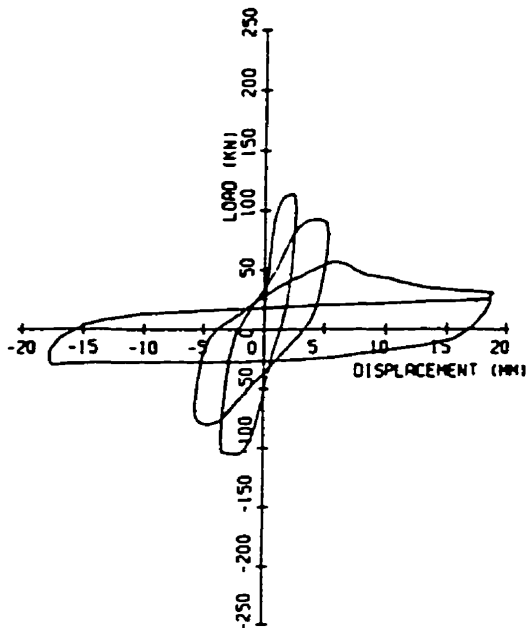
Figure 1.10 (a) Types of connections tested by Spencer (1986)



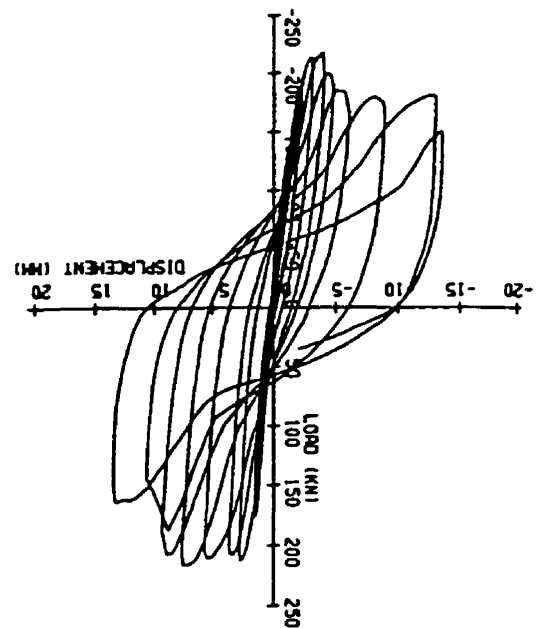
(b) Hysteresis loops for connection Ab9



(c) Hysteresis loops for connection Ar8

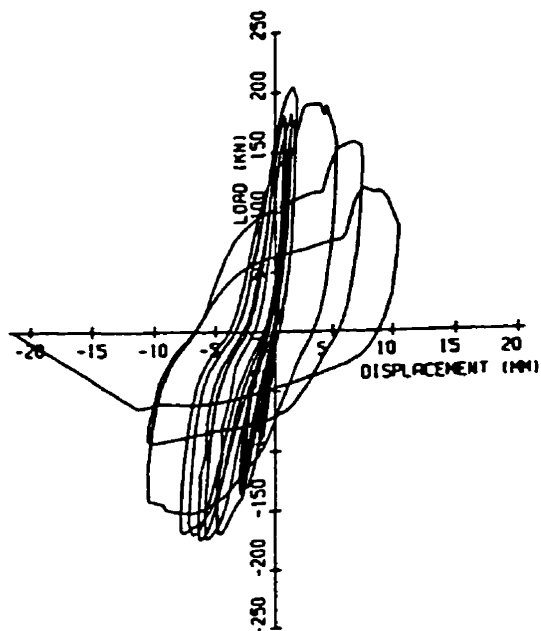


(d) Hysteresis loops for connection B4

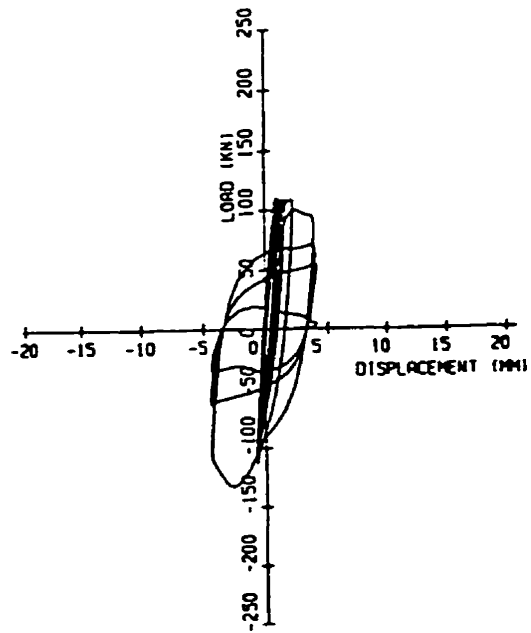


(e) Hysteresis loops for connection Ab19

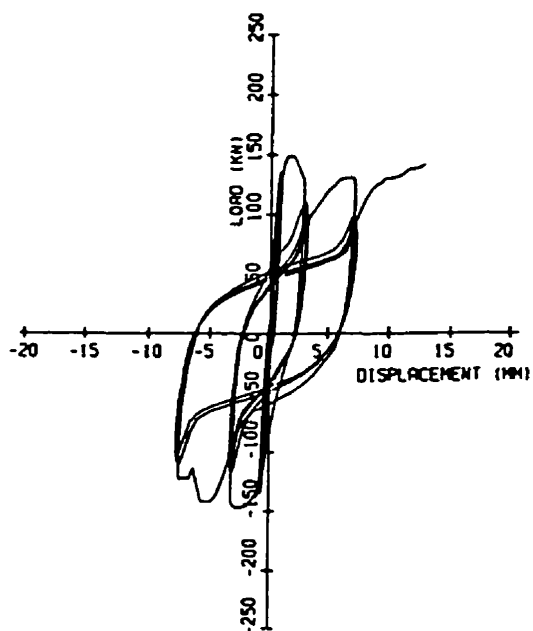
Figure 1.10 (Continued) Behaviour of connections as reported by Spencer (1986)



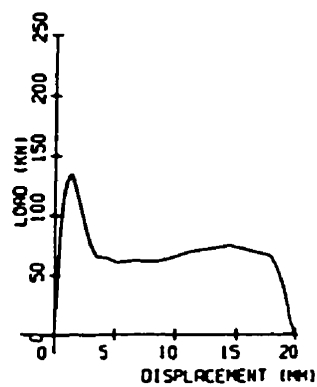
(f) Hysteresis loops for connection Aa2



(g) Hysteresis loops for connection Ab6

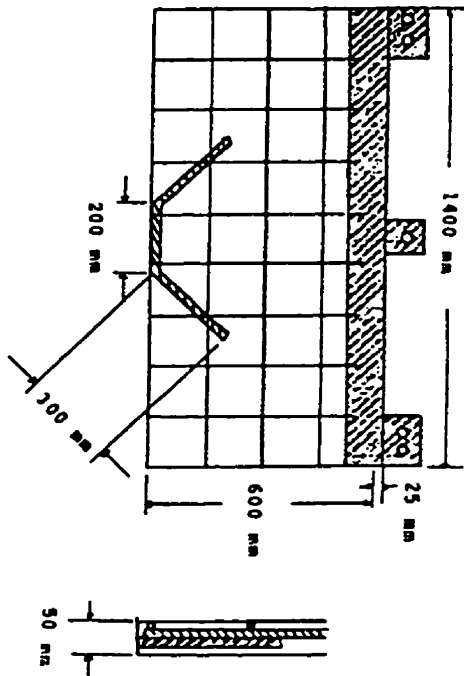


(h) Hysteresis loops for connection As2

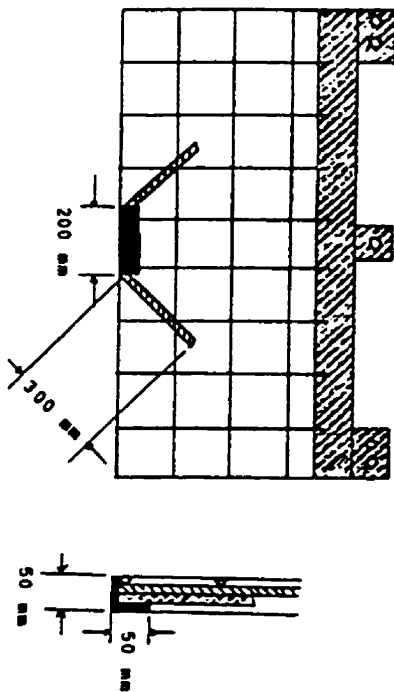
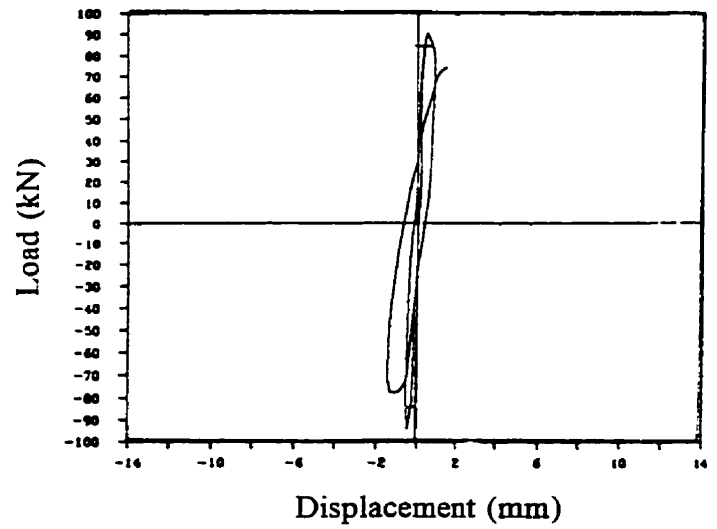


(i) loading curve for connection Ab8

Figure 1.10 (Continued) Behaviour of connections as reported by Spencer (1986)



Rebar #1 at 45 degrees



Rebar #1 at 45 degrees welded to angle

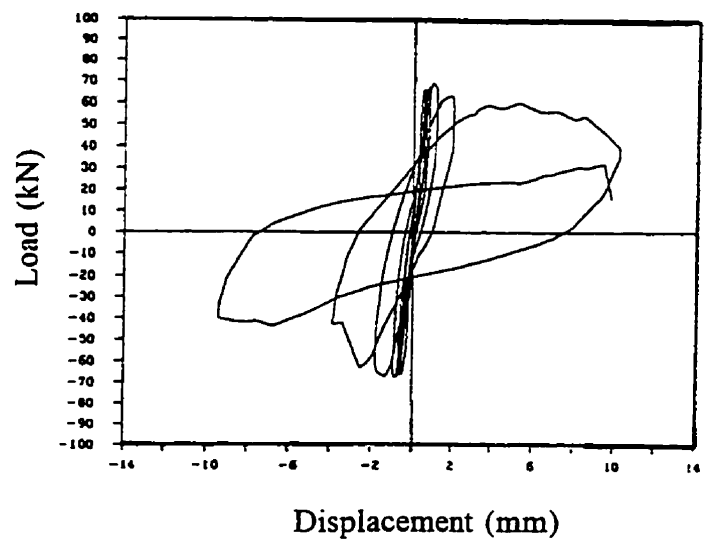
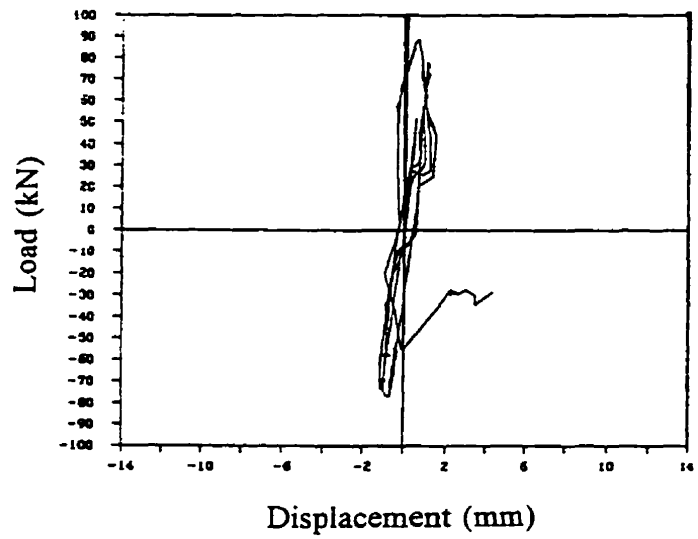
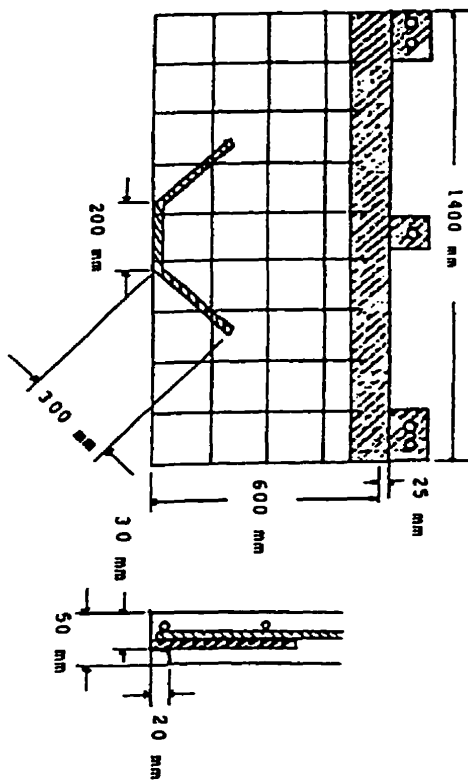
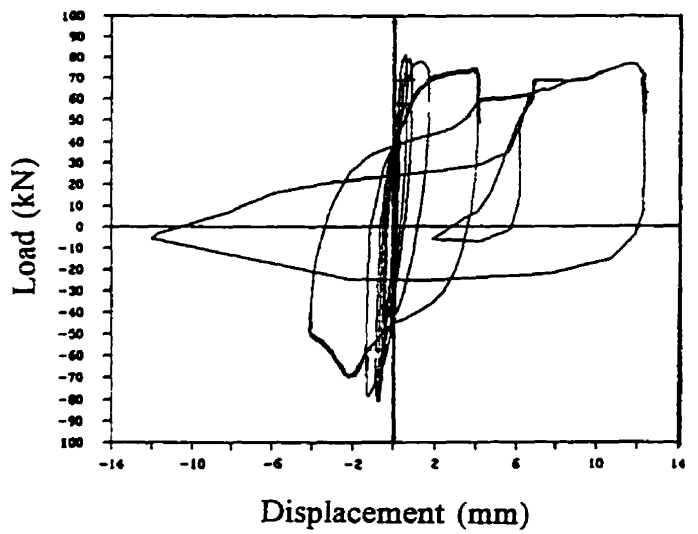
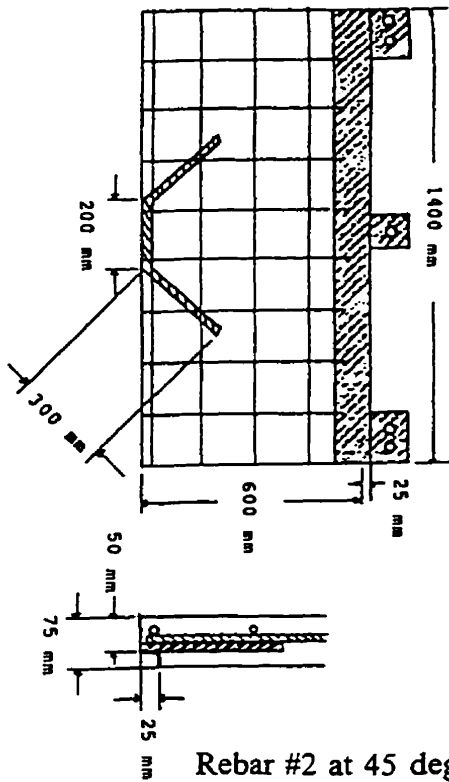


Figure 1.11 Cyclic test on an embedded bar connection as reported by Kallros (1987)

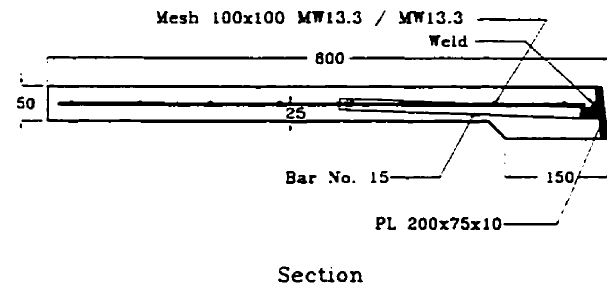
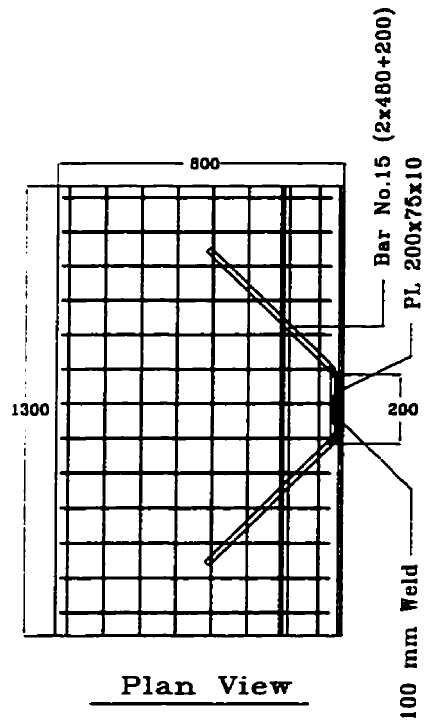


Rebar #2 at 45 degrees with small recess

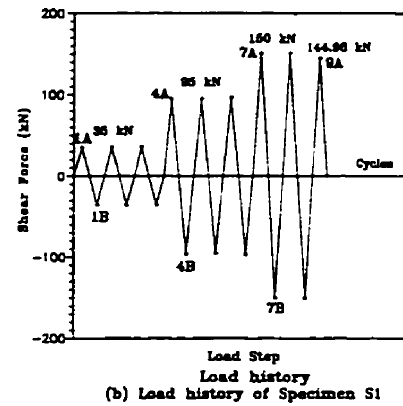
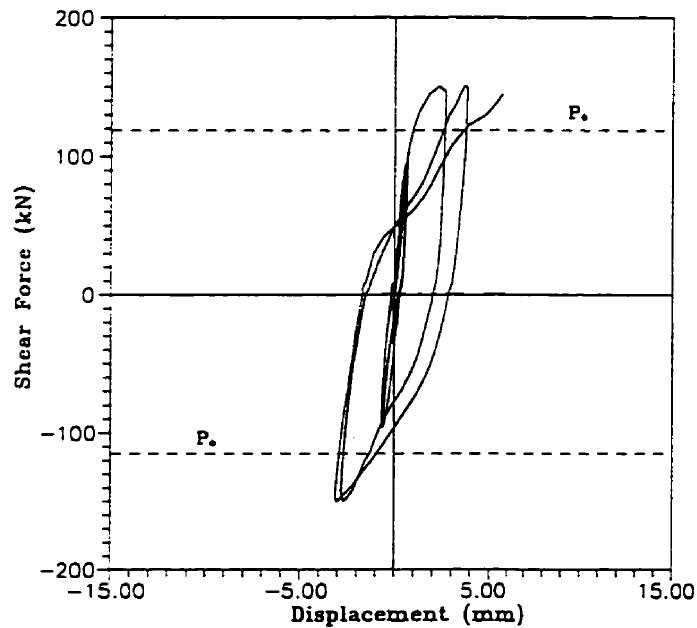


Rebar #2 at 45 degrees with a recess

Figure 1.11 (Continued) Cyclic test on connection as reported by Kallros (1987)

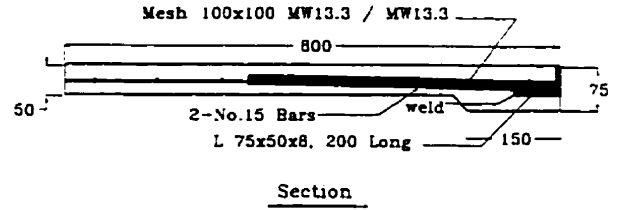
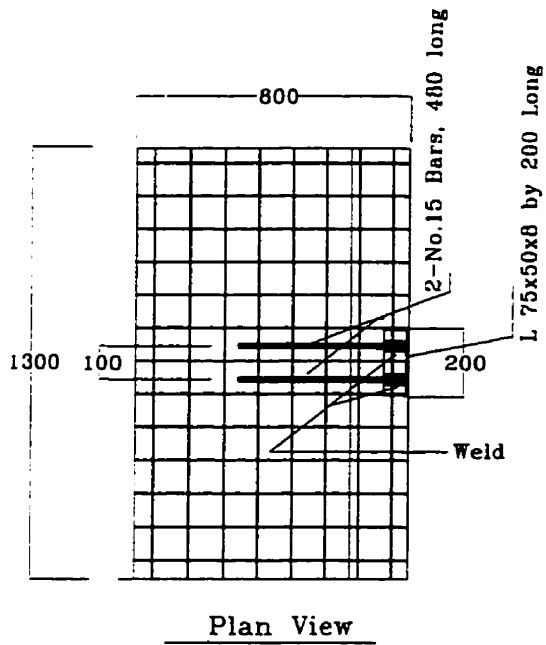


(a) Details of Specimen S1

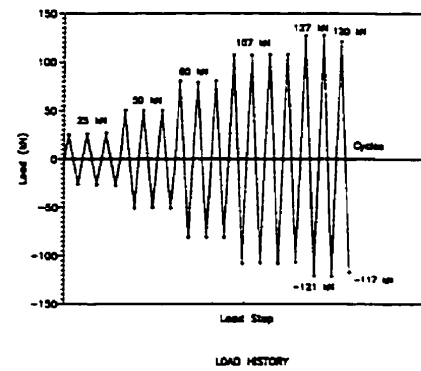
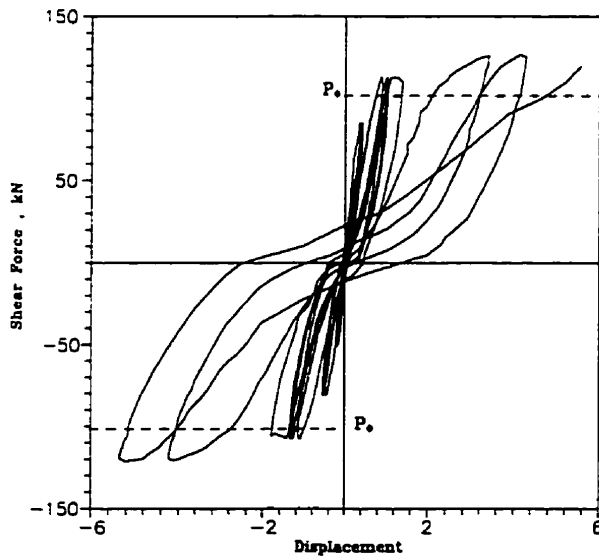


(b) Loading history and hysteresis response

Figure 1.12 Specimen details and load-deflection curve for Connection S1 tested by Shariatmadar et al (1992)

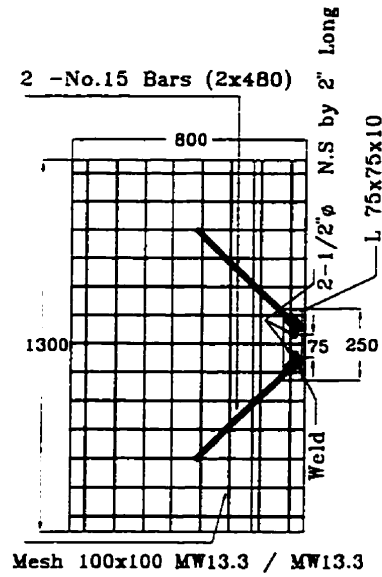


(a) Details of Specimen S3



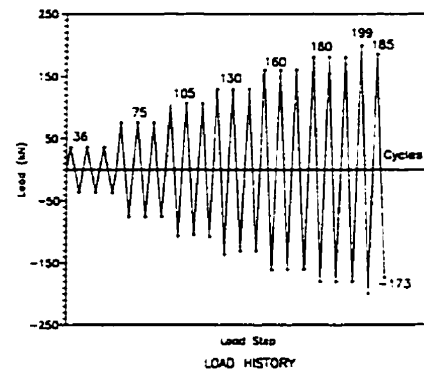
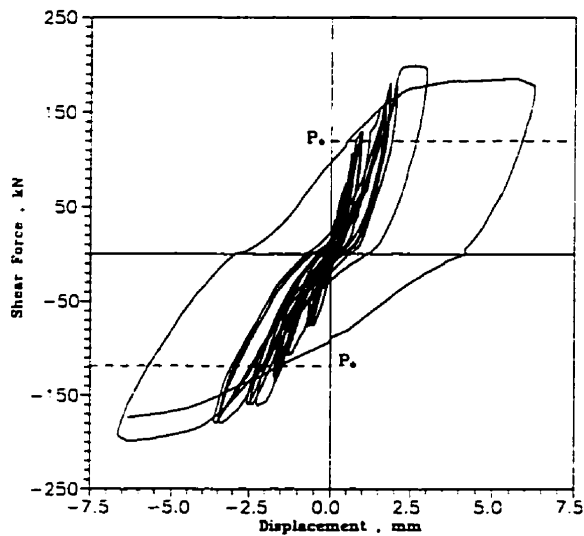
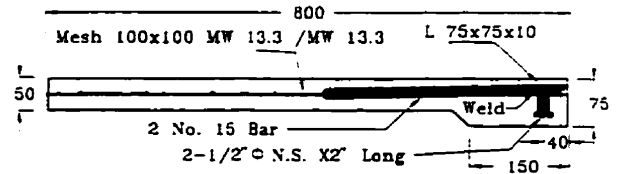
(b) Loading history and hysteresis response

Figure 1.14 Specimen details and load-deflection curve for Connection S3 tested by Shariatmadar et al (1992)



Plan View

(a) Details of Specimen S5



(b) Loading history and hysteresis response

Figure 1.15 Specimen details and load-deflection curve for Connection S5 tested by Shariatmadar et al (1992)

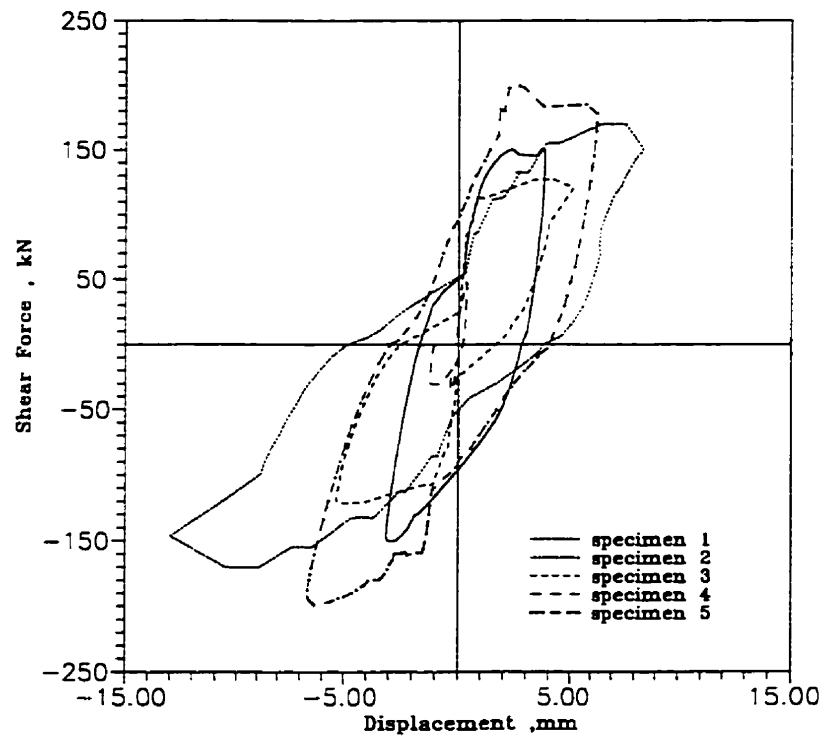


Figure 1.16 Envelopes of load-deflection responses

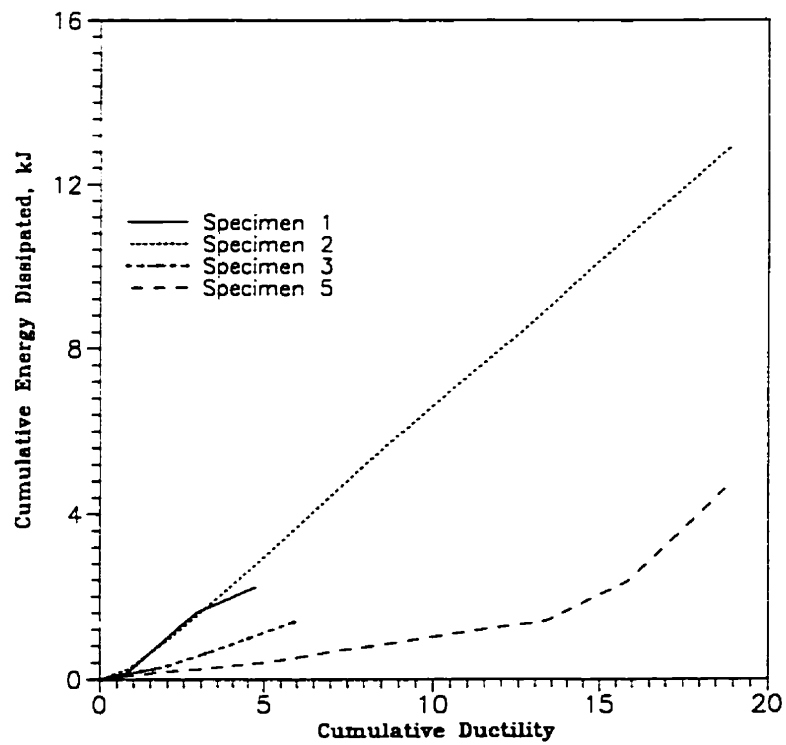


Figure 1.17 Energy dissipated by specimens

1.3 Current Design Approach

1.3.1 Lateral Seismic Force

The National Building Code of Canada (N.B.C.C., 1990) specifies the following two approaches for the evaluation of the design earthquake load on a building:

- (i) Quasi-static seismic analysis
- (ii) Dynamic analysis, using either average response spectra or step - by - step calculation in the time domain of the response of the structure subjected to a particular disturbance.

The quasi-static approach is usually quite satisfactory using the quasi-static seismic analysis. The equivalent elastic base shear, V_e , to determine the minimum lateral seismic force, according to NBCC 1990, is given by:

$$V_e = VSIFW \quad (1-1)$$

where V = zonal velocity ratio

S = seismic response factor (see Figure 1.18)

z_a = acceleration-related seismic zone factor

z_v = velocity-related seismic zone factor

T = fundamental period of vibration of the structure (in seconds)

= $0.1N$ for moment - resisting frames (N = number of storeys)

= $0.09h_n/\sqrt{D_s}$ for other structures (h_n and D_s are the height and length of the lateral load resisting system, respectively)

I = importance factor of the structures.

= 1.5 for post-disaster building, 1.3 for schools and 1.0 for all other buildings

F = foundation factor (ranging from 1.0 for rock to 2.0 for soft deep soil)

W = dead load of the structure plus 25% of the design snow load.

The minimum lateral seismic force, V , for which the structure has to be designed is:

$$V = (V_e / R)U \quad (1-2)$$

where R = force modification factor that depends on the capability of the structure to absorb energy during inelastic response (see Table 1.1).

$U = 0.6$ (calibration factor to adjust the level of protection)

It must be mentioned that precast concrete buildings are not listed in Table 1.1, nor does the Canadian Standard, CSA A23.3-M94 (CSA, 1994) give specific seismic design guidance for precast construction.

In addition to the lateral seismic forces, a building must also be designed for torsional moments in the horizontal plane. The largest design eccentricity, e_n , is given by:

$$e_n = 1.5e + 0.10D_n$$

or

$$e_n = 0.5e - 0.10D_n$$

where e = distance between the centre of mass and centre of rigidity of the structure

D_n = plan dimension of the building in the direction of the computed eccentricity.

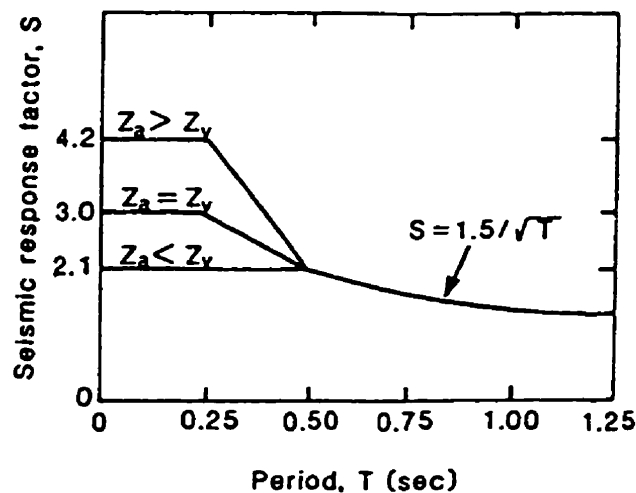


Figure 1.18 Seismic response factor (National Building Code of Canada, 1990)

Table 1.1 Force modification factors for reinforced concrete structures
designed and detailed according to CAN3-A23.3 (N.B.C.C., 1990)

Type of lateral load resisting system	R
-Ductile moment-resisting space frame	4.0
-Ductile flexural shear wall	3.5
-Moment-resisting space frame with nominal ductility	2.0
-Wall with nominal ductility	2.0
-Other lateral force resisting systems	1.5

1.3.2 Connections

Horizontal loads from wind or earthquakes are usually transmitted to shear walls or moment resisting frames through the roof and floors acting as horizontal diaphragms. It is assumed that a diaphragm acts as a deep horizontal beam as shown in Figure 1.19 (CPCI, 1987). It is analogous to plate girders or I-beams. The shear walls or frames are considered to be the supports of this analogous beam. Therefore, the support reactions are induced by the lateral force resulting from the earthquake loading. The tension and compression chord forces are induced in the diaphragm by assuming the bending of the diaphragm in its own plane. The precast concrete double-tees which span parallel to the supporting walls or frames must be designed to transfer the shear between the adjacent elements and also to the supporting elements. Thus the design of a diaphragm is essentially a connection design problem.

1.3.2.1 Connection with Deformed Bar Anchors

This section presents the data and methodology for designing typical flange connections between double-tee flanges. Deformed bars are anchored in the concrete by bond. The required embedded length can be calculated in the same manner as the development length for the normal reinforcing bars. The CPCI Metric Design Manual (CPCI, 1987) suggests the following equations for development length of deformed reinforcing bar anchors:

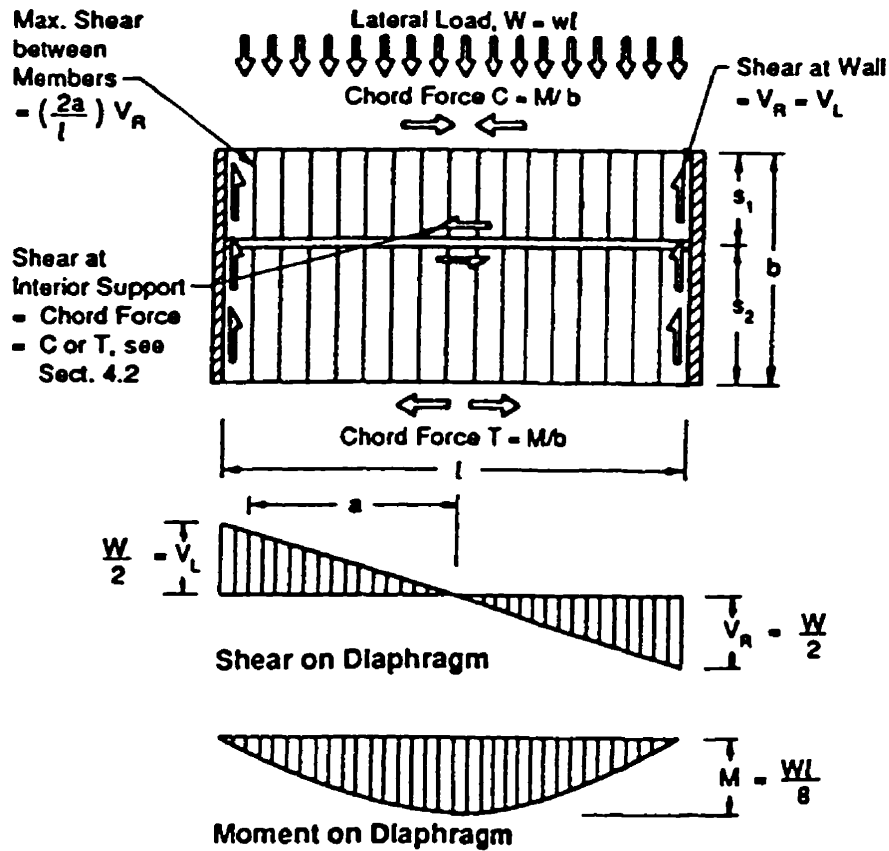


Figure 1.19 Analogous beam design of diaphragm (CPCI, 1987)

For No. 35 reinforcing bars or smaller, in tension:

$$l_d = 0.019 A_s f_y / \sqrt{f'_c} \quad (1-3)$$

which has to be greater than the minimum of $0.058 f_y d_b$ or 300 mm

$$\text{For No. 45 bars,} \quad l_d = 26 f_y / \sqrt{f'_c} \quad (1-4)$$

$$\text{and for No. 55 bars,} \quad l_d = 34 f_y / \sqrt{f'_c} \quad (1-5)$$

while the compression development length is given by :

$$l_d = 0.24 d_b f_y / \sqrt{f'_c} \quad (1-6)$$

where A_s = cross-sectional area of the reinforcing bar (mm^2)

d_b = diameter of the bar (mm)

All modification factors currently in the code would apply.

The ACI Code (ACI 318, 1989) assumes that the basic development length

required to prevent splitting failures, $l_{db\ sp}$, is a function of the cross-sectional area of the bar, while the length required to prevent pull-out, $l_{db\ po}$, is a function of the diameter of the bar. The equations to determine basic development length are as follows:

For No. 35 bars and smaller:

$$l_{dp\ sp} = 0.02A_s f_y / \sqrt{f'_c} \quad (\text{S. I. units}) \quad (1-7)$$

$$l_{dp\ po} = 0.375d_b f_y / \sqrt{f'_c} \quad (\text{S. I. units}) \quad (1-8)$$

The ACI code requires that the basic development length to prevent splitting failure given by equation (1 - 7) be multiplied by a modification factor " k_1 " to account for the clear cover thickness. Both development lengths to prevent splitting and pull-out failures must be multiplied by " k_2 " to account for the modification factors listed in Table 1.2. Therefore, the development lengths to prevent splitting and pull-out failures, respectively, are:

$$l_d\ sp = k_1 k_2 l_{db\ sp} \quad (1-9)$$

$$l_d\ po = k_2 l_{db\ po} \quad (1-10)$$

where $k_1 = 2.0$ for $c \leq d_b$

$k_1 = 1.4$ for $d_b < c < 2d_b$

$k_1 = 1.0$ for $c \geq 2d_b$

$k_1 = 0.8$ for $c_1 \geq 2.5d_b$ in addition to the factors listed below

c = clear cover thickness

c_1 = clear cover thickness to the edge

k_2 = factors listed in Table 1.2

Finally, the development length, l_d , required by the ACI Code is the larger of the two lengths calculated by equations (1-9) and (1-10) but it must not be less than 300 mm. Although the ACI Committee 408 has not explicitly recommended an anchorage length (development length) under cyclic loading, it is likely that the committee would adopt proposals such as those presented by the researchers at the University of Texas at Austin and Cornell University^[20]. The results are summarized as follows:

1) For straight anchorages with a cover of at least $1.5 d_b$:

$$l_d = (1860 \times d_b) / \sqrt{f'_c} > 30 d_b \quad (\text{Imperial units}) \quad (1-11)$$

$$l_d = (154.5 \times d_b) / \sqrt{f'_c} > 30 d_b \quad (\text{S. I. units}) \quad (1-12)$$

ii) For hooked bar anchors:

$$l_{hd} = (1800 \times d_b) / \sqrt{f'_c} \quad (\text{Imperial units}) \quad (1-13)$$

$$l_{hd} = (149.5 \times d_b) / \sqrt{f'_c} \quad (\text{S. I. units}) \quad (1-14)$$

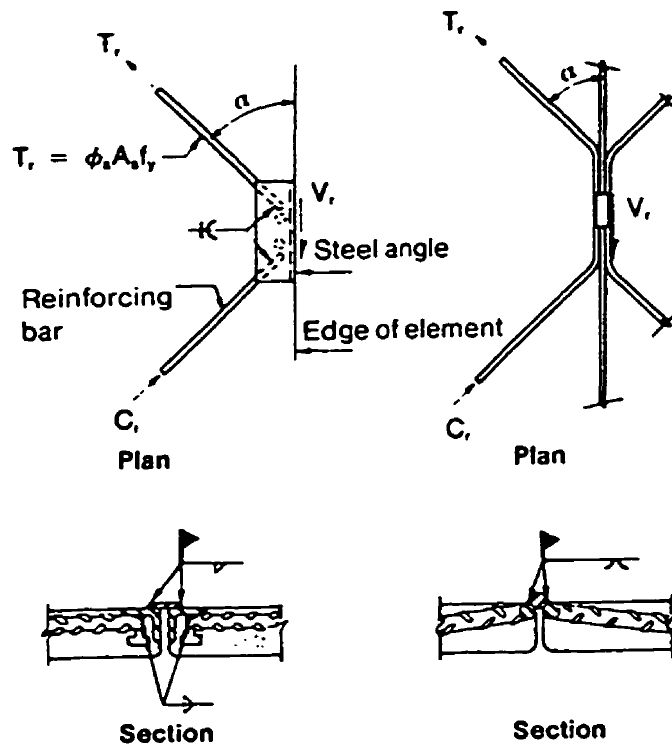
All modification factors currently in the code would apply. It must be mentioned that in this study, the equations for the evaluation of developments lengths of the reinforcing bar subjected to cyclic loading are used (Equations 1-12 and 1-14).

Table 1.2 Values of modification factor k_2 for development length (ACI, 1989)

(a) Top reinforcement - horizontal reinforcement with more than 300 mm of fresh concrete cast below the reinforcement	1.3
(b) Lightweight aggregate concrete, or, if f_{ct} is specified	1.3 $0.556 \sqrt{f'_c} / f_{ct}$
(c) Excess reinforcement	$(A_s \text{ required} / A_s \text{ provided})$

The CPCI Metric Design Manual (CPCI,1987) suggests that this connection should be designed to resist only the shear forces. Since earthquake loading is induced into the diaphragm in any direction, the connections must be designed for the most critical load combination. For convenience, the ATC, the Applied Technology Council, (ATC-3, 1978) requires that 100% of the forces for one principal axis plus 30% of the force for the perpendicular axis be used for design of these connections. In design and analysis , this combination is considered for both principal directions and requires the maximum component strength to be used.

The current method for design of connections utilizes the development length provisions as described above and analysis based on the truss analogy model, as presented by the Canadian Prestressed Concrete Institute (CPCI, 1987) and the Precast/Prestressed Concrete Institute (PCI, 1971, 1985). A typical detail for double-tee flange connections is presented in Figure 1.20.



Notes: (a) Design of welds and weld details in accordance with Canadian Standard Association (CSA-W186 and CSA-G30.16 -M)

Figure 1.20 Typical flange weld plate details (CPCI,1987)

In using the truss analogy, it is assumed that for a given shear force, the forces resulting in the tension and compression legs of the anchorage reinforcing are equal (see Figure 1.20). Therefore, the design shear force for the connection is determined by nodal equilibrium for the truss members as follows:

$$V_r = 2T_r \cos \alpha = 2C_r \cos \alpha \quad (1-15)$$

Therefore,

$$V_r = 2\phi_s A_s f_y \cos \alpha \quad (1-16)$$

In a typical connection, the angle α is equal to 45 degrees, consequently, for a typical connection, the factored shear resistance is given by:

$$V_r = \sqrt{2}\phi_s A_s f_y \quad (1-17)$$

where $\phi_s = 0.85$

A_s = area of the reinforcing bar cross section (mm^2)

f_y = yield strength of the steel bar

In the case when force is applied normal to the connection, the factored capacity of the connection is determined by:

$$V_r = \sqrt{2}\phi_s A_s f_y \quad (1-18)$$

1.3.2.2 Connection with Welded Headed Studs

Welded headed studs are designed to transfer shear, tension or a combination of the two from one element to an adjacent one through the connection. The designs are based on either strength of the steel, or of the concrete, and both must be checked to find the critical capacity of the joint. It is preferable to design the embedded steel anchor to ensure that a tensile failure in the anchor governs. The current design and analysis method are summarized in the following sections for design of connections with headed studs, which are welded to a steel plate and embedded in unconfined concrete. With confining of the concrete, the critical capacity increases, as expected. The thickness of the plates to which the studs are attached, should be at least 2/3 of the diameter of the studs (PCI, 1987).

1.3.2.2.1 Studs in tension

The studs depend on the head to engage a concrete area that causes a cone-type pull-out failure. The method in the CPCI Metric Design Manual (PCI, 1987) assumes a shearing surface and applies a unit shearing strength to that surface area (see Figure 1.21). The factored tensile capacity of the concrete surrounding a headed stud, for failure by a cone-type pull-out, is given by:

$$P_\phi = 0.30\lambda\phi_c\sqrt{f'_c}A \quad (1-19)$$

where $\phi_c = 0.6$

A = effective stress Area

λ = factor defined in Clause 11.2.3. of CAN3 - A23.3

The effective stress area is the projected area of a 45 degree pyramid from the bearing

edge of the anchor (see Figure 1.22). In the case of a stud group, the area of the heads of studs, or the area involved in overlapping stressed areas of the studs must not be included. Also, if the thickness of concrete is not sufficient to develop the full projection area, a reduction in the effective stress area must be considered.

Since the cone type pull-out failure occurs in practice, a reduction factor equal to 0.3 is included in the above formulas to account for the difference between the assumed pyramidal and the conical failure surfaces. The effective stress area for a single headed stud (Figure 1.22) is given by

$$A = (2l_e + 1.58d_b^2) - 2.5 d_b^2 \quad (1-20)$$

where l_e = embedded length of the stud

d_b = stud anchor diameter

If the stud is located near a free edge of a member, a bursting type of failure can be expected. In this respect, the minimum cover (mm) from the anchor head to any free edge is given by:

$$m = d_b \sqrt{\frac{f_u}{6\sqrt{f_c}}} \geq 50mm \quad (1-21)$$

where f_u = ultimate tensile strength of the anchor steel

The minimum concrete thickness for the full development of a stud group is:

$$h_{min} = l_e + b/2 \quad (\text{see Figure 1.23}) \quad (1-22)$$

If the above criteria are not satisfied, special procedures are necessary to include a reduction factor. Referring to Figure 1.24, the effective stress area (A) is given by:

$$A = h(2a + 8l_e + 2b - 4h) - A_h \quad (1-23)$$

where A_h = total area of the anchor heads. Other symbols are shown in Figure 1.24.

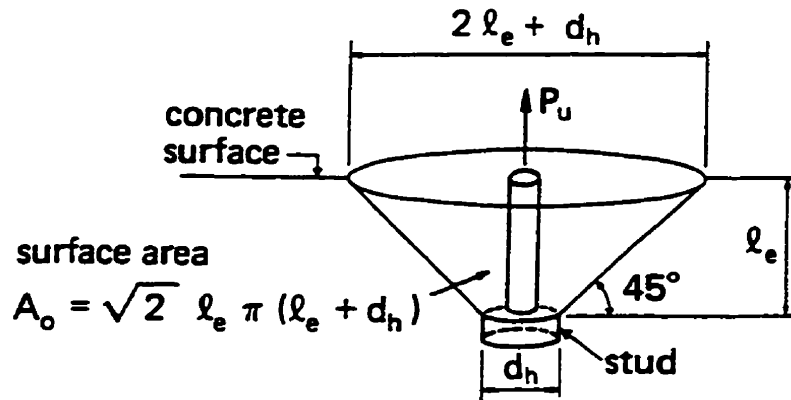


Figure 1.21 Assumed failure surface for headed studs (Martin, 1982)

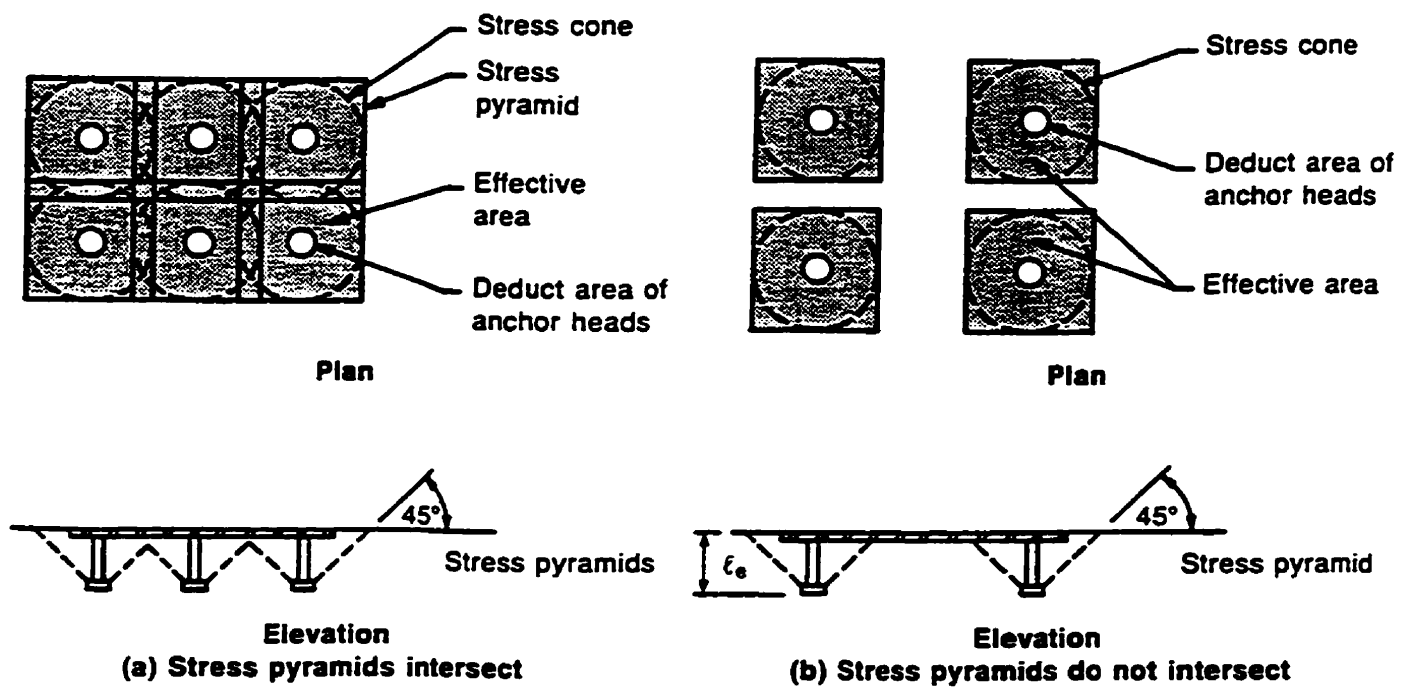


Figure 1.22 Effective stress area for welded headed studs (CPCI, 1987)

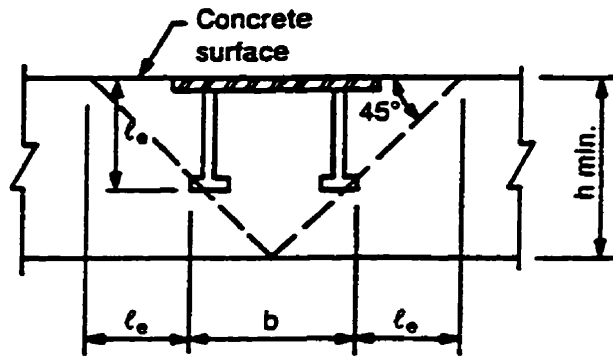


Figure 1.23 Development of welded headed studs (CPCL, 1987)

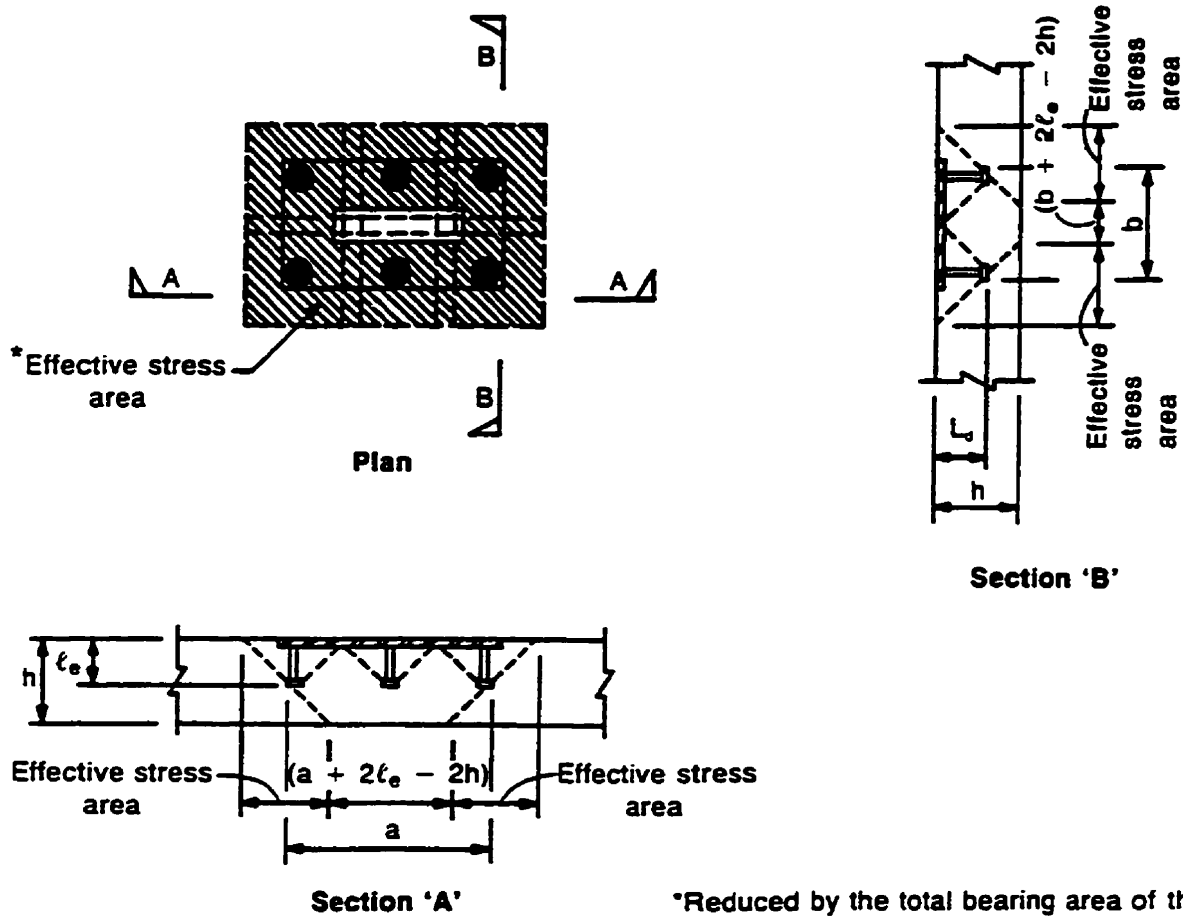


Figure 1.24 Stress area reduction for limited depth A_r , (CPCL, 1987)

When the tensile strength of steel governs, the factored resistance, P_ϕ , of a headed stud is given by:

$$P_\phi = A_b \phi_s f_u \quad (1-24)$$

where A_b = area of stud anchor

$$\phi_s = 0.85$$

f_u = ultimate tensile stress

A value of 400 MPa is recommended for the ultimate tensile strength of the headed stud steel. As mentioned previously, to achieve ductile behaviour, the factored resistance of the connection derived based on steel strength, should not be less than the factored resistance of the connection, derived based on the pull-out failure of the headed stud (concrete cone-type failure).

1.3.2.2.2 Studs in shear

The studs are frequently subjected to shearing forces in precast element connections. The concrete strength for the studs not located near a free edge can be calculated using either the lateral bearing of the stud, or by using the shear friction concept. The factored bearing resistance, B_ϕ , of a single headed stud subjected to shear can be determined using the equation:

$$B_\phi = 1.4 \phi_c A f_c' \quad (1-26)$$

where $\phi_c = 0.6$ and A = projected bearing area of the stud ($d \times l_c$)

The factored shear resistance calculated using the shear friction principle for studs which are welded to a plate is given by:

$$V_\phi = \phi_s A_{vf} f_y \mu \quad (1-27)$$

where $\phi_s = 0.85$

A_{vf} = cross-sectional area of the studs

f_y = yield strength of the stud

$\mu = 0.6\lambda$ for contact plane coincidental with the concrete surface

$\mu = 0.8\lambda$ for contact plane a full plate thickness below concrete surface

λ = factor defined in Clause 11.2.3. of CAN3 - A23.3

The above value of V_ϕ must be smaller than the values calculated using Equations 11.31

and 11.32 and provisions of Section 11.7 of the CSA Standard A 23.3-M94 (CSA, 1994). When the shear load is in the direction of a free edge, the shear strength of concrete becomes more critical. The factored shear resistance specified in the CPCI Design Manual (CPCI, 1987) can be calculated using the equation:

$$V_{\phi} = 0.30\lambda\phi_c\sqrt{f'_c}A \quad (1-28)$$

where A = effective stress area.

1.3.2.2.3 Studs in combined shear and tension

Most of the embedded steel devices in precast concrete connections are normally subjected to a combination of shear and tensile forces. For design purposes, an interaction equation for concrete can be used as follows:

$$\frac{1}{\phi_1} \left[\left(\frac{P_u}{P_c} \right)^2 + \left(\frac{V_u}{V_c} \right)^2 \right] \leq 1.0 \quad (1-29)$$

where $\phi_1 = 0.85$

P_u = factored tension loads

V_u = factored shear loads

P_c = tensile strength of the connection governed by concrete failure

V_c = shear strength of the connection governed by concrete failure

For steel, the interaction equation is:

$$\frac{1}{\phi_2} \left[\left(\frac{P_u}{P_s} \right)^2 + \left(\frac{V_u}{V_s} \right)^2 \right] \leq 1.0 \quad (1-30)$$

where $\phi_2 = 1.0$

P_s = tensile strength of the connection governed by steel failure

V_s = shear strength of the connection governed by steel failure

It must be noted that in the current method for the design of connections between double-tee flanges under lateral loading, is based only on the shear acting on the connection.

1.4 Objectives

Although there are empirical methods available for the design of these connections under static loads, these methods are unable to predict the complete response of a given connection. The Precast/Prestressed Institute (PCI, 1971, PCI, 1985 and CPCI, 1987) design method provides a model only for the elastic design of connection, however, this method cannot predict the inelastic response under a monotonically increasing load, or the inelastic response under reversed cyclic loads. Furthermore, the present design procedure is based only on the shear force and ignores the axial forces induced in the connection during an earthquake.

The experimental work on connections between precast elements completed by Shariatmadar (1992) and by Shariatmadar and Mirza (1995), and the analytical work by Mirza and Rasouli (1991) and Berhe et al (1992) and some recorded data (Kunze, 1965) from the response of the prefabricated structures subjected to earthquakes show presence of significant axial forces (compressive or tensile) in the connections between the double-tee roof elements, specially in the vicinity of rigid frames, stiff structural walls and cores supporting the roof diaphragm in the structural system. The present design practice ignores these axial forces, and therefore, there is a risk of failure of these connections, or of severe cracking in the vicinity of these joints during a major earthquake. Consequently, the existing design procedures must be upgraded to account for these axial forces.

The principal objective of this research program is to demonstrate that the connections in thin concrete panels must be designed for the combined shear and axial forces induced by seismic loading, via a reversed cyclic loading interaction relationship. The goal of this research is to develop appropriate design procedures for such connections in precast concrete structures.

The experimental-analytical program consists of investigations into the following domains:

- i. Determination of the boundary conditions for the test specimen (with a single connection) from the results of analysis of a complete flange of a double-tee. This

will be achieved by matching the stress conditions in the vicinity of the connections with the stress conditions in the complete flange of the double-tee. Furthermore, an initial nonlinear finite element analysis is conducted to study the response of connections subjected to combined shear and tensile forces. This improves the understanding of the influence of the tensile forces (in addition to the shearing forces) on the behaviour of the connection.

- ii. Experimental study of the complete response of selected connections (anchor bars welded to the plate at angles of 45° , and with 90° hooks at the ends, headed studs welded to a steel angle and appropriate combinations of the above anchorage systems) subjected to:

- a) monotonically increasing shearing forces,
- b) reversed cyclic shearing forces,
- c) reversed cyclic axial forces,
- d) reversed cyclic combinations of shearing and axial forces,

all increasing gradually in stages until failure (using displacement control). The experimental data will consist of load-deformation, steel and concrete strain characteristics, crack width and crack propagation, energy absorption characteristics, and the mode of failure of the specimen. In addition, the experimental results will be used to develop anchorage reinforcement patterns aimed at augmenting the strength, stiffness, ductility and energy absorption characteristics of the connections.

- iii. The effect of reversed cyclic axial forces on the overall response of the connections, especially the shear response is investigated, and suitable shearing force - axial force interaction diagrams and equations are developed and derived to be used as tools for design of connections subjected to combined shear and axial loads.

- iv. The current design procedures for the connections are developed to include the

effect of reversed cyclic loading (simulation of the seismic loads), and the effect of axial forces normal to the connection.

- v. Nonlinear finite analysis of the complete behaviour of the connections using the NONLACS (NONLinear Aalysis of Concrete and Steel Structure) Program. The connection are subjected to monotonically increasing in-plane shear and tensile axial loads and their combinations to establish a new simple procedure to evaluate the response of the connections and to provide an analytical interaction relationship with an adequate level of confidence. Moreover, this helps to derive proper analytical interaction equations suitable for design and analysis of the connections under an earthquake loading, which results in considerable savings in the computation time and cost for the analysis of connections subjected to reversed cyclic loading.
- vi Comparison of the experimental and analytical responses of the connection subjected to monotonically increasing shear forces to demonstrate the accuracy of the connection modelling in the NONLACS Program. The experimental results from the monotonic loading are also compared with the results using the empirical method to evaluate the accuracy of the current design procedures. Some alternative methods are developed for design and analysis of the connections subjected to static loading.
- vii. The final stage in this experimental-analytical study consists of development of practice-oriented methods to determine the shearing and axial forces at the joints between the precast roof elements when subjected to earthquake loads, and a simple procedure to design these connections.

Chapter 2

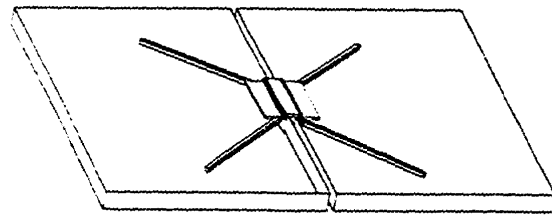
Experimental Program

2.1 Introduction

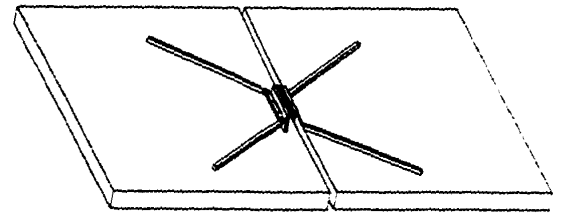
Thirty-one specimens of six different connection types between double tee flanges proposed to be used in the precast industry, were investigated in this research program. The connection details are shown in Figure 2.1, and were selected for their simplicity and ease of construction. A total of thirty one precast concrete specimens in concrete panels of size 1300 mm x 800 mm x 50 mm thick, were constructed and tested under the monotonically increasing shearing forces, and combined reversed cyclic shear and axial forces in the Jamieson Structures Laboratory at McGill University.

The connections details are defined as follows:

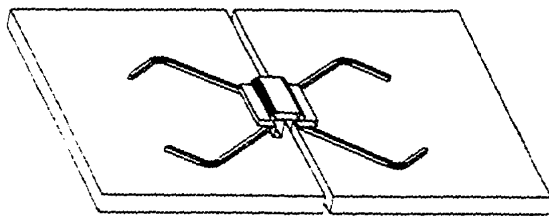
- 1) Connection Type A: Specimens with a bent deformed bar anchor welded to the flange of a steel angle plate, see Figures 2.1a and 2.11.
- 2) Connection Type Ap: Specimen with a bent deformed bar anchor welded to the steel plate, as shown in Figures 2.1b and 2.12.
- 3) Connection Type B: Specimens with a bent deformed bar anchor and a 90° hook welded to the steel angle flange (Figure 2.1c and 2.13).
- 4) Connection Type C: Specimens with two straight deformed bar anchors, oriented at 90° and welded to an angle flange, Figures 2.1d and 2.14.
- 5) Connection Type D: Specimens with three deformed bar anchors with two oriented at 45° and the remaining one at 90° and all welded to the flange of steel angle, see Figures 2.1e and 2.15.
- 6) Connection Type E: Specimens with headed studs welded to the steel plate, as shown in Figures 2.1d and 2.16.



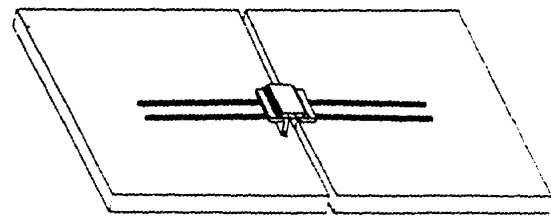
(a) Connection Type A



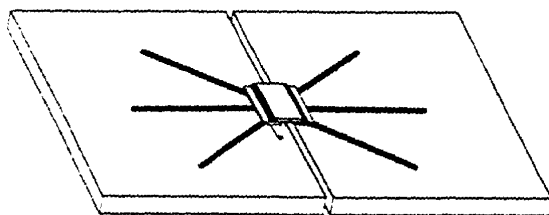
(b) Connection Type Ap



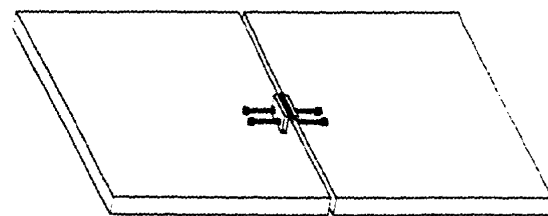
(c) Connection Type B



(d) Connection Type C



(e) Connection Type D



(f) Connection Type E

Figure 2.1 Details of connections investigated in this research program

2.2 Design of Prototype Building

To understand the importance of connections between the flanges of double-tee slabs and between the double-tee flange and the lateral load resisting elements, and the loads transmitted by these connections, the design of a single story precast structure with a precast double-tee roof and precast wall panels is reviewed in the following sections.

2.2.1 Single - Storey Precast Structure

Figure 2.2 shows the plan and elevation of a prototype single - storey precast concrete structure. This example of the prototype structure is designed and analyzed using the National Building Code of Canada (N.B.C.C. 1994), the Canadian Standard Association (CSA A23.3-M94) and the CPCI Metric Design Manual (1987).

The precast building, shown in Figure 2.2, is 38.4 m by 48 m in plan and has 4.8 m clear height inside. The walls are composed of 400 mm deep double-tees and the roof assemblage of 600 mm deep double-tees. The double-tees are 2400 mm wide with stems 1200 mm on centres. The flanges of the roof tees are 50 mm thick, while the wall tees are 100 mm thick.

The seismic design of the prototype structure was performed in the North - South direction, and it was assumed that the building was located in Montreal. The dead and live loads conform to the 1990 National Building Code of Canada and are:

Wall Panels:

$$\text{Unit weight of wall panel} = 3.3 \text{ kN/m}^2$$

$$\text{Height of wall panel} = 6150 \text{ mm}$$

North and South Walls:

$$\text{Total length of one wall} = 48.0 \text{ m}$$

$$\text{Weight above mid - height} = (3.45)(48.0)(3.3) = 545 \text{ kN}$$

$$\text{Weight of complete wall} = (6.15)(48.0)(3.3) = 971 \text{ kN}$$

East and West Walls:

$$\text{Total length of one wall} = 38.4 \text{ m}$$

$$\text{Weight above mid - height} = (3.45)(38.4)(3.3) = 436 \text{ kN}$$

$$\text{Weight of the complete wall} = (6.15)(38.4)(3.3) = 770 \text{ kN}$$

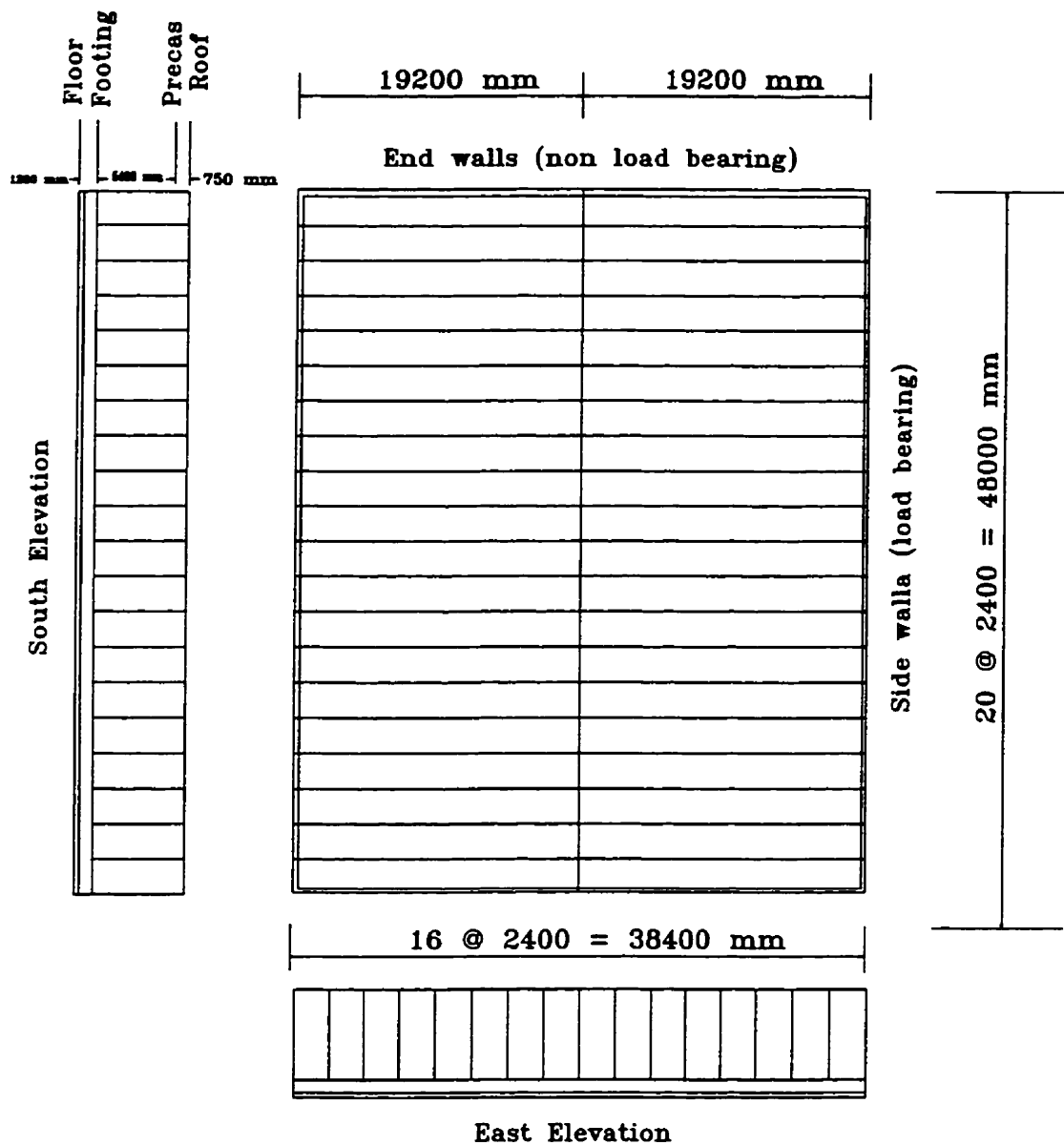


Figure 2.2 Details of single-storey precast wall panel building

Roof:

Weight of a roof panel (with water proofing membrane) = 2.8 kN/m^2

Weight of roof = $(48.0)(38.4)(2.8) = 5179 \text{ kN}$

Weight of building (roof + all walls) = 8675 kN

Snow Load:

25% of design snow load, S , is taken into consideration.

For the structure under consideration, the snow load was calculated to be:

$$S = 2.64 \text{ kN/m}^2$$

$$\text{Total snow load} = (0.25)(38.4)(48.0)(2.64) = 1216 \text{ kN}$$

Total lumped weights:

$$\text{- Roof + N-S walls + Snow} = 8926 \text{ kN}$$

$$\text{- Roof + N-S wall + E-W walls + Snow} = 9933 \text{ kN}$$

Specified lateral seismic force

The minimum lateral seismic force, acting at the top of the panels is given:

$$V = (v \ s \ I \ F \ W)(U) / (R)$$

where the seismic factors are:

$$v = 0.1 \text{ for Montreal}$$

$$T = 0.1 \text{ N, for one storey building } N=1, \text{ therefore, } T = 0.1 \text{ s}$$

$$S = 4.2 \text{ for } T < 0.25 \text{ s}$$

$$I = 1.0$$

$$F = 1.0$$

$$W = \text{weight of the roof, N-S walls, and 25\% of design snow load} = 8926 \text{ kN}$$

$$U = 0.6$$

$$R = \text{force modification factor} = 2.0$$

Therefore,

$$V = (0.1 \times 4.2 \times 1.0 \times 1.0 \times 8926 \times 0.6) / (2.0) = 1125 \text{ kN}$$

The design earthquake load, V , is applied to the roof diaphragm as indicated in Section 1.3.1. To guard against accidental torsional effects, a minimum eccentricity equal to 10% of the lateral dimension must be assumed between the centroid of resistance and the centre of mass as shown in Figure 2.3. Therefore, the design forces on the roof diaphragm are:

Maximum shear force, V_w , assumed to act on each of the east and west walls at the roof level is : $V_w = 0.6 V = 0.6 \times 1125 = 675 \text{ kN}$

$$\text{Average shear intensity, } v_w = 675/38.4 = 17.6 \text{ kN/m}$$

$$\text{Maximum bending moment, } M_w = (V_w \times 0.6L)/2 = (675 \times 0.6 \times 48)/2 = 9720 \text{ kN.m}$$

$$\text{Maximum chord forces, } C_w = T_w = M_w/d = 9720 / 38.4 = 253 \text{ kN}$$

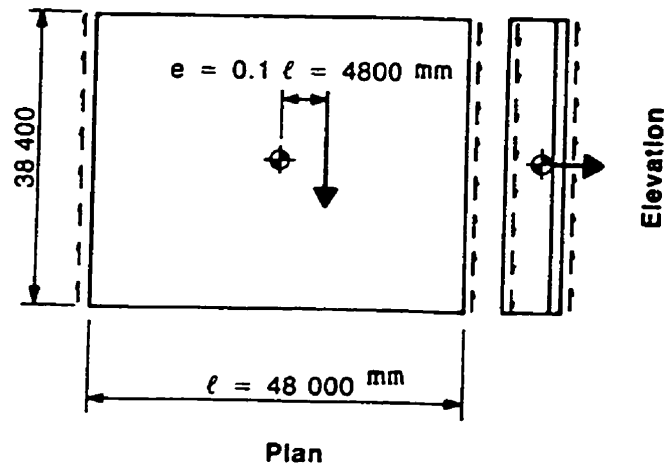


Figure 2.3 Specified forces acting on roof and walls

Resistance analysis

Analysis of the structure is undertaken in two parts. The first part deals with design and detailing of the roof diaphragm and its connections, while the second part deals with the design of the wall panels. Because this thesis focuses on the behaviour of the connection between the flanges of the double-tees which constitute the roof diaphragm and between the flange and lateral load resisting elements, focus is placed on the design of the connections.

The diaphragm is required to resist the shear forces induced by the earthquake load. The maximum factored shear intensity in double tee flanges adjacent to the wall panel is :

$$v_f = 1.5 v_w = 1.5 \times 17.6 = 39.6 \text{ kN/m}$$

where the v_f is the shear force per unit length of the structure in North-South direction. The Applied Technology Council (ATC, 1978) requires an additional 30% of the shear force to be considered perpendicular to the axis of the shear load (tensile axial force) for design of the connections. Therefore, for design and analysis purposes, the combination of both in-plane forces must be considered. Then, the maximum factored axial intensity is:

$$t_f = 0.3 \times v_f = 39.6 \times 0.3 = 11.9 \text{ kN/m}$$

where the t_f is shear force per unit length.

The design shear strength of the concrete per unit length in the double tee flanges along the web is:

$$\begin{aligned} v_c &= 0.2 \phi_c \sqrt{f'_c} d \times 1000 \\ &= 0.2 \times 0.6 \sqrt{35} \times 50 \times 1000 = 35.5 \text{ kN/m} \end{aligned}$$

which is less than v_f , therefore the flanges need reinforcement, normally using welded wire fabric (WWF). The principles of shear friction are used to design the diaphragm shear reinforcement. The steel area required to resist the v_f and t_f is determined as follows:

$$A_b = (v_f) / (\mu \phi_s f_y) + (t_f) / (\phi_s f_y)$$

where $\mu=1.25$, for concrete cast monolithically [CSA A23.3-M94, Clause 11.7.4.3(a)]

A_b = area of required reinforcement per unit length normal to the web,

and

$$f_y = 350 \text{ MPa (assumed)}$$

Substitution of the values in the above equation gives:

$$\begin{aligned} A_b &= (v_f) / (\mu \phi_s f_y) + (t_f) / (\phi_s f_y) = (39.6) / (1.25 \times 0.85 \times 350) + (11.9) / (0.85 \times 350) \\ &= 135 \text{ mm}^2/\text{m} \end{aligned}$$

Therefore, the double-tee flanges are reinforced with WWF 100 x 100 MW 13.3 / MW 13.3 which provides a steel area, $A_s = 133 \text{ mm}^2/\text{m}$ in each direction.

Therefore, the shear and tensile resistances of the double tee flanges is adequate. Figure 2.4 shows the reinforcement details of flange.

Design of Connections

The roof and the wall to the roof connections must now be designed for the applied loads along the east and west edges. The factored design shear and tensile forces are $V_f = v_f \times 38.4 = 39.6 \times 38.4 = 1520 \text{ kN}$ and $T_f = t_f \times 38.4 = 11.9 \times 38.4 = 457 \text{ kN}$. A typical connection to be used in this design is shown in Figure 1.13. The material resistance factor for reinforcing steel, $\phi_s=0.85$ is used to find the force in the reinforcing steel bar. According to the CSA Standard, CAN3-A23.3-M94, Section 21.2.3.2, if these

connections are intended to yield during an earthquake, an allowance is made for the loss of the strength during the cycles of inelastic loading by using an additional member resistance factor, $\phi_m=0.7$. In this study, the most critical case is considered, since the purpose of research program is to investigate the strength of the connection in the inelastic range, subjected to reversed cyclic loads.

For the case designed to resist both shear and tension, the area of the steel required is additive. If the compression force in the compression leg and the tensile force in the tensile leg are C_r and T_r , respectively, and the factored shear and tensile resistances are provided in accordance with the truss analogy principles for Connection Type A, the total area of steel bar is calculated as follows:

Utilizing the force equilibrium in the connection of the resisting forces and applied factored tensile and shear loads, gives:

$$T_{rc} \cos 45^\circ + C_{rc} \cos 45^\circ = V_f \quad (1)$$

$$T_{rc} \cos 45^\circ - C_{rc} \cos 45^\circ = T_f \quad (2)$$

Solving the above equations for T_{rc} , gives:

$$T_{rc} = (T_f + V_f) \cos 45^\circ \quad (3)$$

and substituting $T_{rc} = n \phi_m \phi_s f_y A_s$ in equation 3 , the total required steel area is:

$$n A_s = [(T_f + V_f) \cos 45^\circ] / (\phi_m \phi_s f_y) \quad (4)$$

where T_f = factored tension force

V_f = factored shear force

$\phi_m = 0.85$

$\phi_s = 0.70$

f_y = yield strength of the reinforcing steel bar

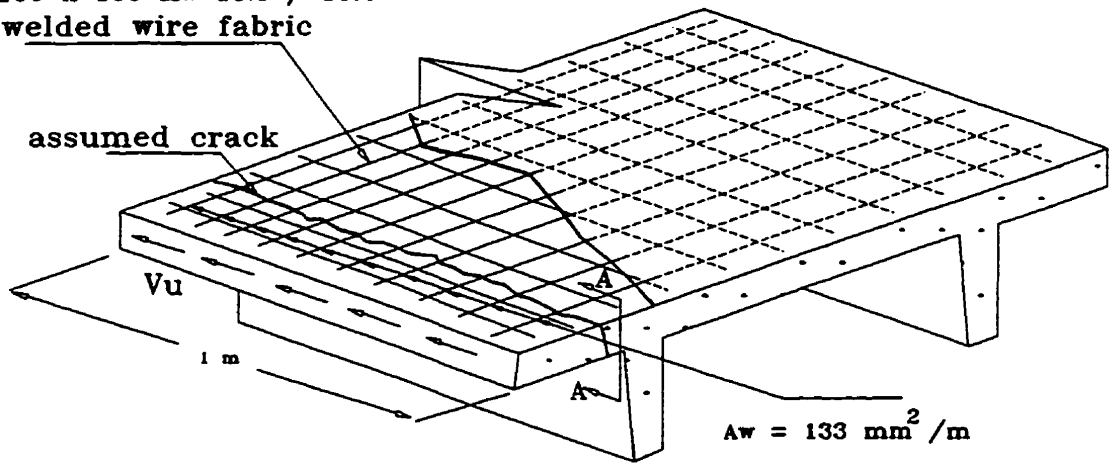
A_s = one leg area of the reinforcing steel bar

n = number of the connection along the flange edge

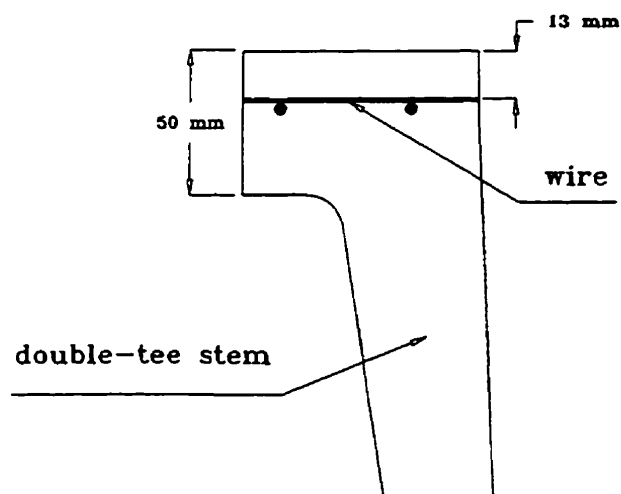
Once the number of connections are determined, the connection spacing can be selected.

100 x 100 MW 13.3 / 13.3
welded wire fabric

assumed crack



(a) Plan



(b) Section A-A

Figure 2.4 Reinforcement details of double-tee flange

Connection between double-tee flange and wall panels

The connection to the east and the west walls (Fig. 2.2) must be designed for the factored design load induced at the top of the walls. The values of V_f and T_f at the top of the walls are 1520 kN and 456 kN, respectively. Substituting these values in equation 4 gives:

$$\begin{aligned} n A_s &= [(T_f + V_f) \cos 45^\circ] / (\phi_m \phi_s f_y) \\ &= (1520 + 456) \cos 45^\circ \times 10^3 / (0.7 \times 0.85 \times 400) = 5871 \text{ mm}^2 \end{aligned}$$

Conventionally, the minimum size of deformed steel bar used as anchors for the connection is M15, with $A_s = 200 \text{ mm}^2$. Therefore, the number of connections is:

$$n = 5871 / 200 = 30$$

The spacing, S , between the connections then must not be exceeded;

$$S = 38400 / 30 = 1280$$

A suitable spacing between the connections would be 1200 mm on centres. The connection details are shown in Figure 2.5.

Connections between double-tee flanges

The double tee flanges must also be connected to each other along their edges using a connection similar to that between the double-tee flange and the wall. The number of connections required is proportional to the shear force which decreases towards the middle of the diaphragm.

During an earthquake, one wall panel may yield while the other walls still be in the elastic range due to the result of torsion and other effects. Therefore, as recommended in the CPCI Metric Design Manual (CPCI, 1987), the resultant shear force on the diaphragm is assumed to act with a minimum eccentricity of 10% relative to the centre. The shear force therefore has zero value at $0.6L$ ($= 0.6 \times 4800 = 28800 \text{ mm}$), see Figure 2.6. Using the shear force distribution shown in Figure 2.6, the connection spacing along the interior flange of the first double tee can be calculated as follows:

The shear force acting on the interior flange:

$$V_{if} = (1520)(28800 - 2400) / (28800) = 1393 \text{ kN}$$

Considering 30 % of the shear force in the axial direction (ATC, 1987) gives:

$$T_{lf} = 0.3 V_{lf} = 1393 \times 0.3 = 418 \text{ kN}$$

Substituting the values of T_{lf} and V_{lf} in equation 4 gives:

$$\begin{aligned} n A_s &= [(T_f + V_f) \cos 45^\circ] / (\phi_m \phi_s f_y) \\ &= (1293 + 418) \cos 45^\circ \times 10^3 / (0.7 \times 0.85 \times 400) = 5380 \text{ mm}^2 \end{aligned}$$

Using M15 anchors for the connection, $A_s = 200 \text{ mm}^2$, the number and the spacing of the connection at the interior flange are given by:

$$n = 5380 / 200 = 27$$

$$S = 38400 / 27 = 1422 \text{ mm}$$

A suitable spacing between connections would be 1400 mm on centres. The connection details are shown in Figure 2.5.

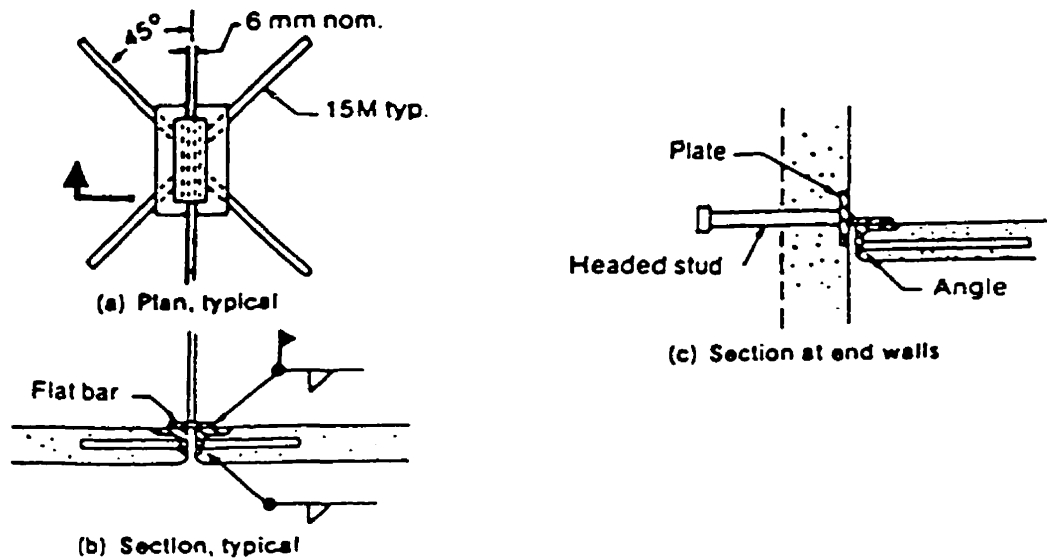


Figure 2.5 Connections between the flanges of double- tees

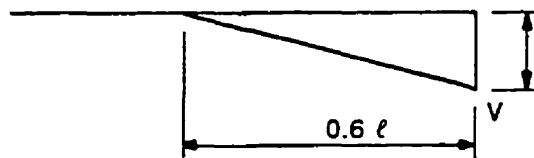


Figure 2.6 Distributed shear on diaphragm with 10% eccentricity

2.3 Choice of Test Specimen Size and Boundary Conditions

To study the connection between the flanges of the double-tees, the ideal solution would be to have a specimen consisting of two connected double-tees and apply the in-plane axial and shear forces to the system to investigate the behaviour of the connections. However, tests of this kind are extremely cumbersome and expensive. Therefore, it was decided to study the behaviour of a single connection in an effective part of the flange resisting the combined axial and shear force applied at the connections. To determine the size of such an effective flange part and its boundary conditions, for simplicity a part of a double-tee flange with three connection was isolated and analyzed using the SAP90 program (Wilson et al, SAP 90, 1989). This would ensure that the stresses in the vicinity of the central connection are reasonably close to those in a full double-tee flange, as the connections further away do not have any significant effect on the central connection.

2.3.1 Analysis of Partial Flange with Three Connections

The in-plane shear and axial forces acting at the connection are transmitted in the plane of the diaphragm which has a large in-plane rigidity at the flange-web connection. The resistance of the connection can be resolved into two components, one parallel to the span of the double-tee and the other perpendicular to it. The resistance to the moments caused by the connection in-plane forces arises from the rigid flange-web connection. To account for these conditions, and for simplicity, the finite element model shown in Figure 2.7, with hinges at each node along the line LM, was adopted.

The analysis of the selected partial flange was made using the computer program SAP90, which employs the stiffness method of analysis. The static analysis of a structure involves the solution of a system of linear equations represented by:

$$K U = R$$

where K is the stiffness matrix, U is the vector of the resulting displacements, and R is the vector of applied loads.

The concrete in the selected partial flange is modeled using 1248 three dimensional 4-node isoparametric membrane shell elements (Figure 2.7). This element has two in-plane translational degrees of freedom (u and w) and a rotational component

in the direction normal to the plane of the element ($\odot_z$). Six shell elements are used to model the welded steel angle, or the steel plate, while 54 beam elements model the steel anchor. The 1352 nodes are numbered in an efficient way to get an optimum stiffness matrix with a minimum band width. The ratio of smallest length to thickness for the flange is $800/50 = 16$, therefore using the shell element represents an appropriate choice. Also, these membrane elements can model the behaviour of slab adequately since the applied loads are only in the plane of the flange. The beam element can model the axial and dowel behaviour of the steel anchor. The loads were applied for different combination of shear (V) and tensile axial forces (T) which were calculated based on the factored ultimate strength of the connection.

The results for principal stresses for the area around the middle connection (region ABDC in Figure 2.7) for a T/V ratio equal to one are shown in Figure 2.8. In Figure 2.8, the negative and positive signs represent the compressive and tensile stresses, respectively.

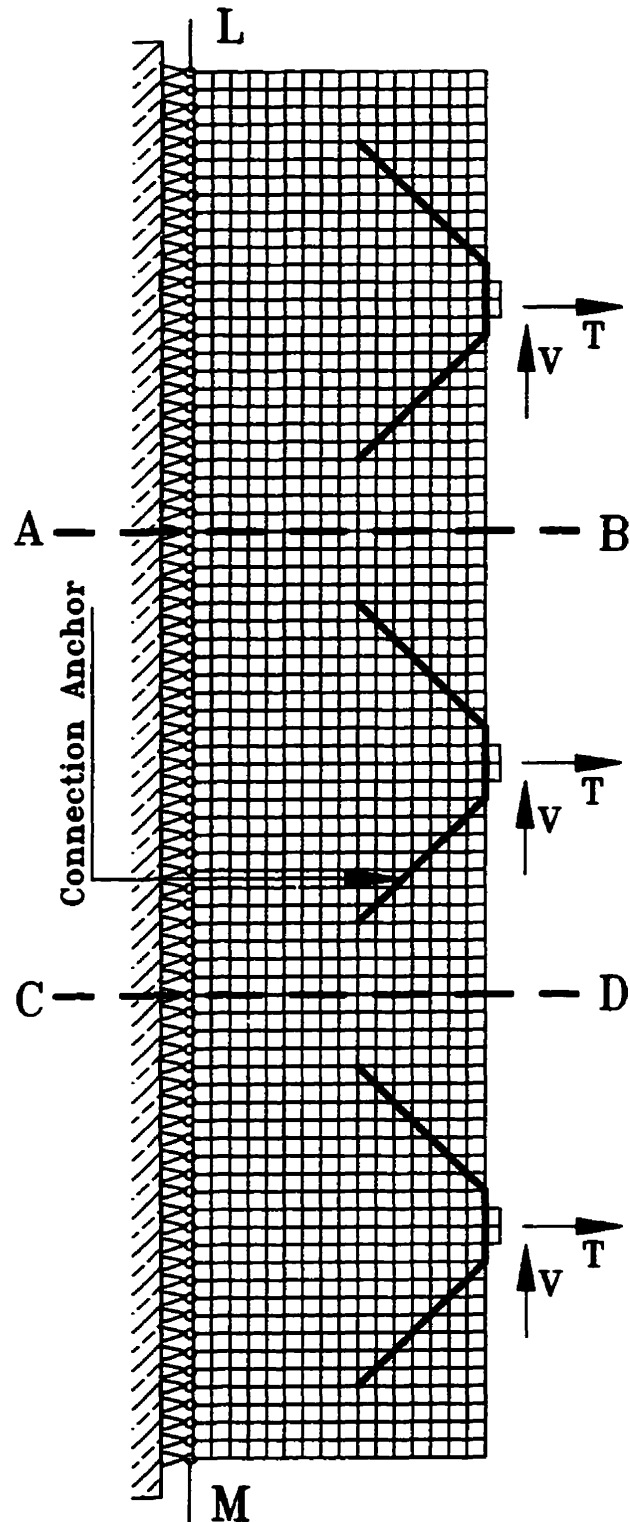


Figure 2.7 Finite element idealization for partial double-tee flange

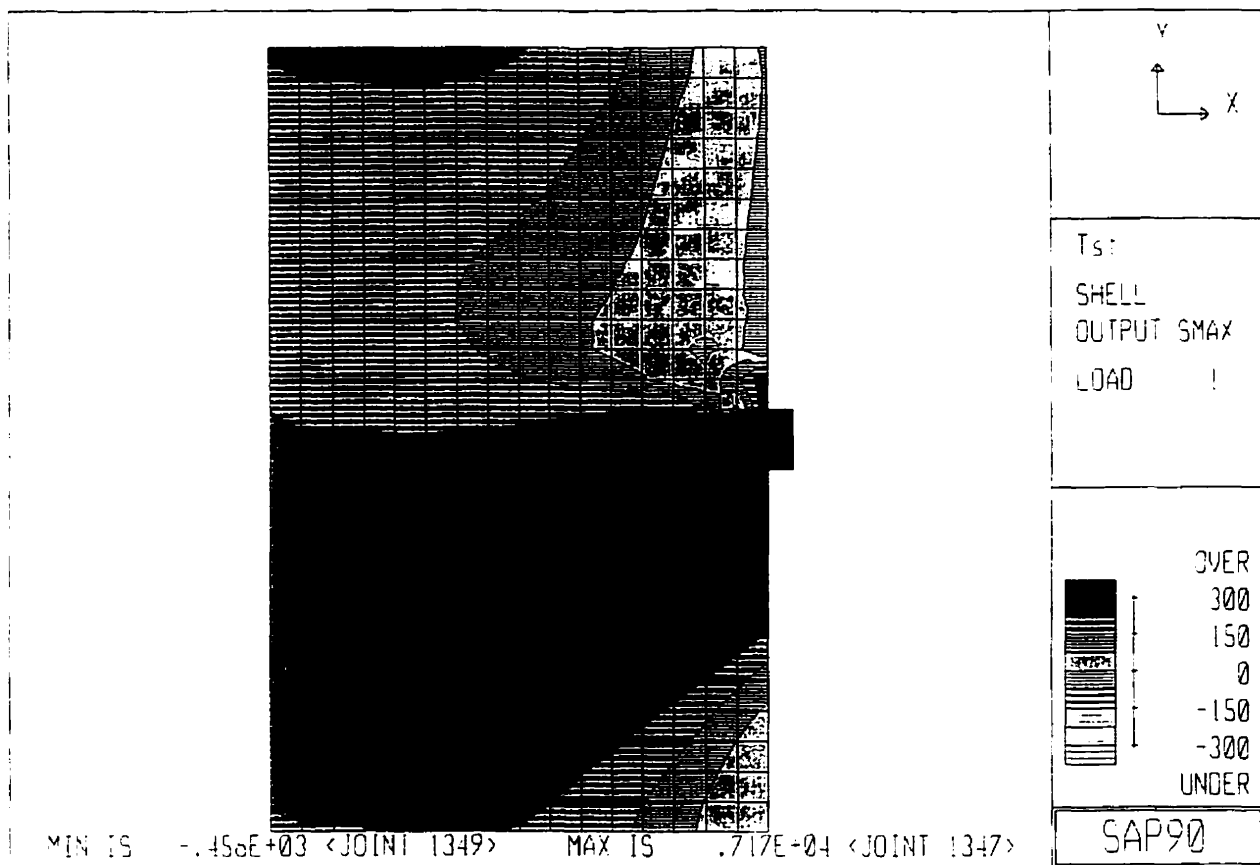


Figure 2.8 Distribution of principal stresses around the middle connection of the partial flange for $T/V=1$ (units = MPa)

2.3.2 Test Specimen and Boundary Conditions

The basic geometry of the test specimen was selected based on the zone of influence of the connection in the partial flange of the double-tee, namely, the region ABDC as shown in Figure 2.7, which is 1300 mm x 800 mm.

The concrete panel of the test specimen and the connection steel plate or angle were modelled using 416 and 2 membrane shell elements, respectively. The reinforcing steel bars were idealized using 18 beam elements. The finite element idealization for the test specimen is shown in Figure 2.9.

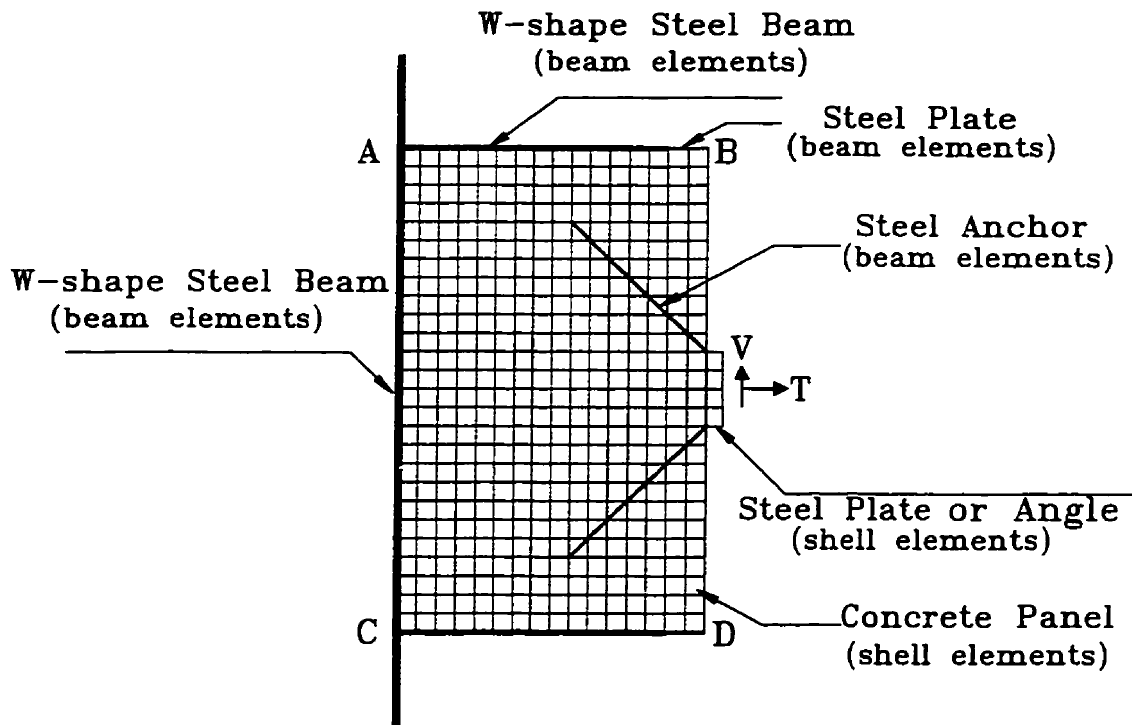


Figure 2.9 Finite element idealization for test specimen

The test specimen boundary conditions were selected to ensure that the stress distribution in the test specimen was as close as possible to that obtained from the analysis of partial double-tee flange with three connections. After some trials, it was decided to use 140 mm by 10 mm steel plates with 1/4" diameter Nelson studs, cast along the specimen boundaries AB, CD and AC (Figure 2.9). W-shaped steel beams (200 x 31 mm) with a 160 mm x 8 mm plate, 800 mm long, welded on both faces of the beam web were bolted to the plate with studs on boundaries AB and CD. The plate with the studs on boundary AC was similarly connected by bolts to a heavy wide flange steel beam (W 310 x 118), which was in turn anchored (post-tensioned) to the laboratory strong floor. These boundary elements were modelled using 16 beam elements for each of the faces AB and CD, and 28 beam elements for the plate on boundary AC.

The details of the finite element model are shown in Figure 2.9, which was analyzed using the SAP90 program. Loads with different combination of shear and axial (T/V ratios) were applied to the connection as shown. The resulting distribution of the

principal stresses in the concrete panel for T/V ratio =1 are shown in Figure 2.10, where the negative and positive signs indicate the compressive and tensile stresses, respectively. Comparison of these principal stresses with those in Figure 2.8 shows an excellent agreement between the two sets of results. Therefore, the above boundary conditions were selected for the subsequent experimental work. Practical requirements and procedure of construction of test specimen in the laboratory are presented in Sections 2.5 (Construction Sequence) and 2.7 (Test Set-up).

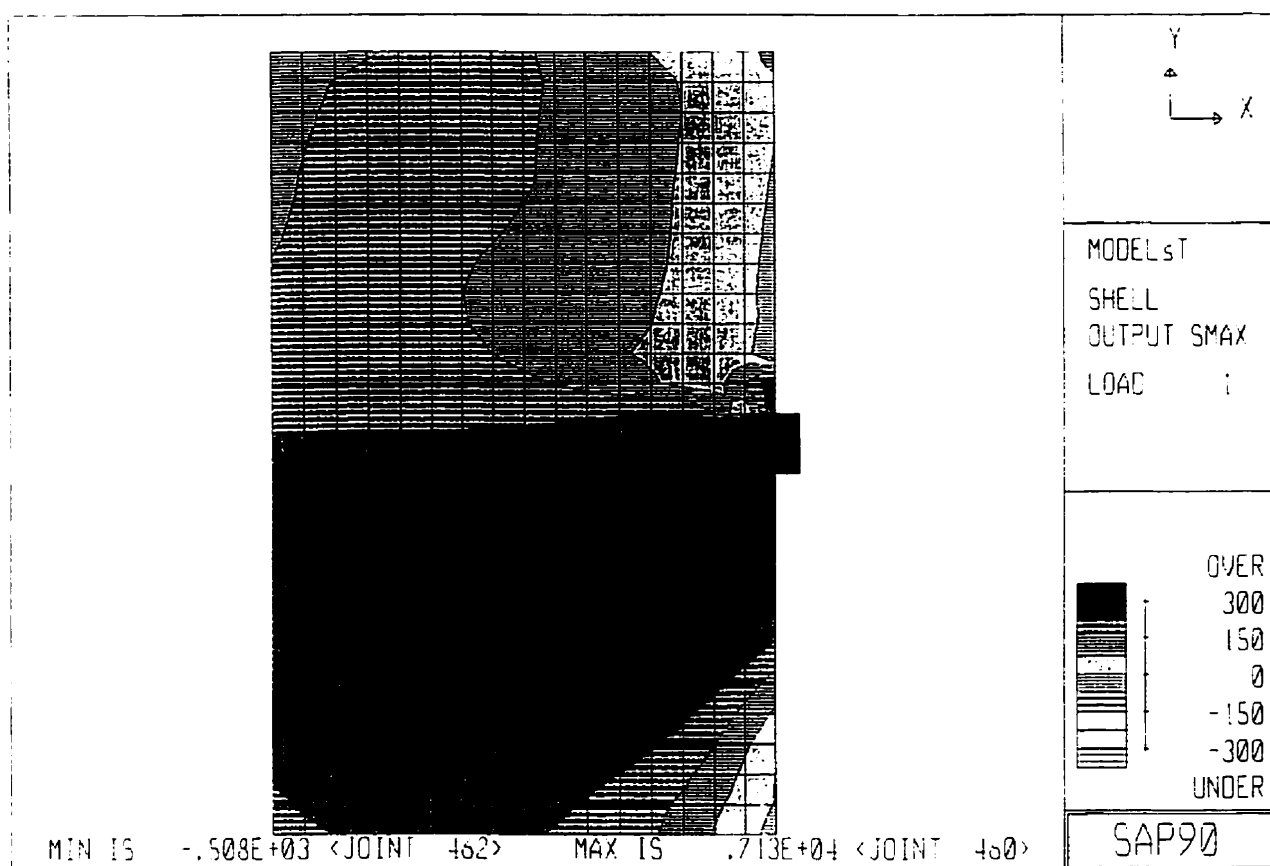


Figure 2.10 Distribution of principal stresses in the test specimen for T/V=1 (units = MPa)

2.4 Design and Description of Test Specimens

The experimental program consisted of testing thirty one specimens for six different types connections useful for the industry (Figure 2.1). Each specimen was 1300 mm by 800 mm in size and 59 mm thick, as modelled for the analysis. These connections were detailed and tested to evaluate their response to monotonically increasing and low-cyclic loading, aimed at determining their ultimate strength , level of ductility and energy absorbing capability, etc.

For connections using deformed reinforcing bar under cyclic loads, most of the load is carried by bond between the reinforcing steel and the concrete . The tensile and the compressive strengths of the concrete, cover thickness and the amount of transverse reinforcement play a dominant role in controlling the connection behaviour. Bond failure modes under low cyclic loading are very similar to those under monotonic loading. Under low - cycle loading, two types of bond failure are typical. The first is a direct pull-out of the bar, which occurs when ample confinement is provided to the bar. The second type of failure is a splitting of the concrete where the cover or confinement is insufficient. In this case, failure is primarily due to the radial tensile stresses caused by the bearing of the bar lugs. In this study, since the cover thickness is small, a splitting tensile failure will limit the bond capacity for a given anchorage length to a value lower than that for failure by pull-out action. This effect is more important for cyclic than for monotonic loading, because of the increased severity of cracking and a more rapid deterioration of bond under cyclic loading. To compensate for the smaller cover thickness and the resulting concrete splitting, the anchorage length is increased to 480 mm instead of the normally required length of 300 mm. With the increased embedment length, it is expected that the bar will yield in tension before it pulls out or splits the surrounding concrete. In this respect, the results of the investigations of the anchorage lengths of reinforcing bars subjected to cyclic loads at the University of Texas at Austin and Cornell University (ACI, 1990) are used. The ultimate capacity of the connection under monotonically increasing load (referred to as "monotonic strength") is calculated based on these development lengths. For small concrete cover thickness, determination of the

ultimate strength under monotonic load is based partially on the work of Jimenez et al (1979), which in turn depends on the concrete cover thickness. The specimens were designed initially using the simple truss analogy and the "shear friction" concept described in the CPCI Metric Design Manual (CPCI, 1987). All test specimens were initially designed using the material resistance factors $\phi_c = 0.60$ and $\phi_s = 0.85$ and assuming a concrete compressive strength of $f'_c = 35$ MPa, a reinforcing bar yield strength, $f_y = 400$ MPa, and an ultimate strength of steel headed stud, $f_u = 400$ MPa. The dowel action of the anchor bar was considered. The influence of the wire fabric mesh was not included. It should be noted that the final ultimate strength of the connections used for comparison with the test results were calculated using the theoretical calculations, CPCI method and the NONLACS program based on the measured material properties. The design calculations are presented in Appendix.

2.4.1 Test Specimen Series A

Specimens in Series A (Connection Type A) contained No. 15 weldable deformed reinforcing bars used as the anchorage in the connection, bent at 45 degrees, with an inside bend diameter of 95 mm. The length of the straight part and each inclined leg were 200 mm and 480 mm, respectively. The straight part and the part of the inclined legs were welded to a 75 mm x 50 mm x 10 mm thick steel angle, 350 mm long (Figure 2.11). The test specimens contained uniform wire fabric mesh, 100 x 100 MW 13.3/ MW 13.3, with a steel ratio $\rho = 0.0027$ in each direction. The mesh was placed in the central plane of the 50 mm thick slab. The specimen had an average clear concrete cover thicknesses of 20.5 mm. The monotonic ultimate strengths were determined using the "truss analogy". The ultimate capacity of each truss leg depends on the yield strength of the anchor leg. The details of calculation are presented in Appendix.

2.4.2 Test Specimen Ap

Specimen Ap (Connection Type Ap) was identical to the specimens in Series A,

except that the 200 mm straight part of the reinforcing bar was welded to a 50 x 10 mm thick, 250 mm long inclined steel plate instead of the steel angle. Details of the connection is shown in Figure 2.12.

2.4.3 Test Specimen Series B

Specimens in Series B (Connection Type B) contained a No. 15 weldable deformed reinforcing bar anchor which was bent at 45 degrees. A right angled hook was provided at the end of the straight lead of the bar as shown in Figure 2.13. The length of the straight part of each leg was 400 mm plus twelve bar diameters for the hook. The 200 mm straight part and a part of the leg were welded to a 75 x 50 x 10 mm thick steel angle, 350 mm length. The use of 90° hook enabled the connection to remain in the inelastic range for a larger number of cycles before failure. The welded wire mesh used to reinforce the panel was identical to that for the Connection Type A. The concrete clear cover was similar to that for the Connection Type A. The details of Connection Type B are shown in Figure 2.13.

The specimen was designed with special consideration for hooked reinforcing bars subjected to cyclic loading. Detailed calculations are presented in Appendix A.

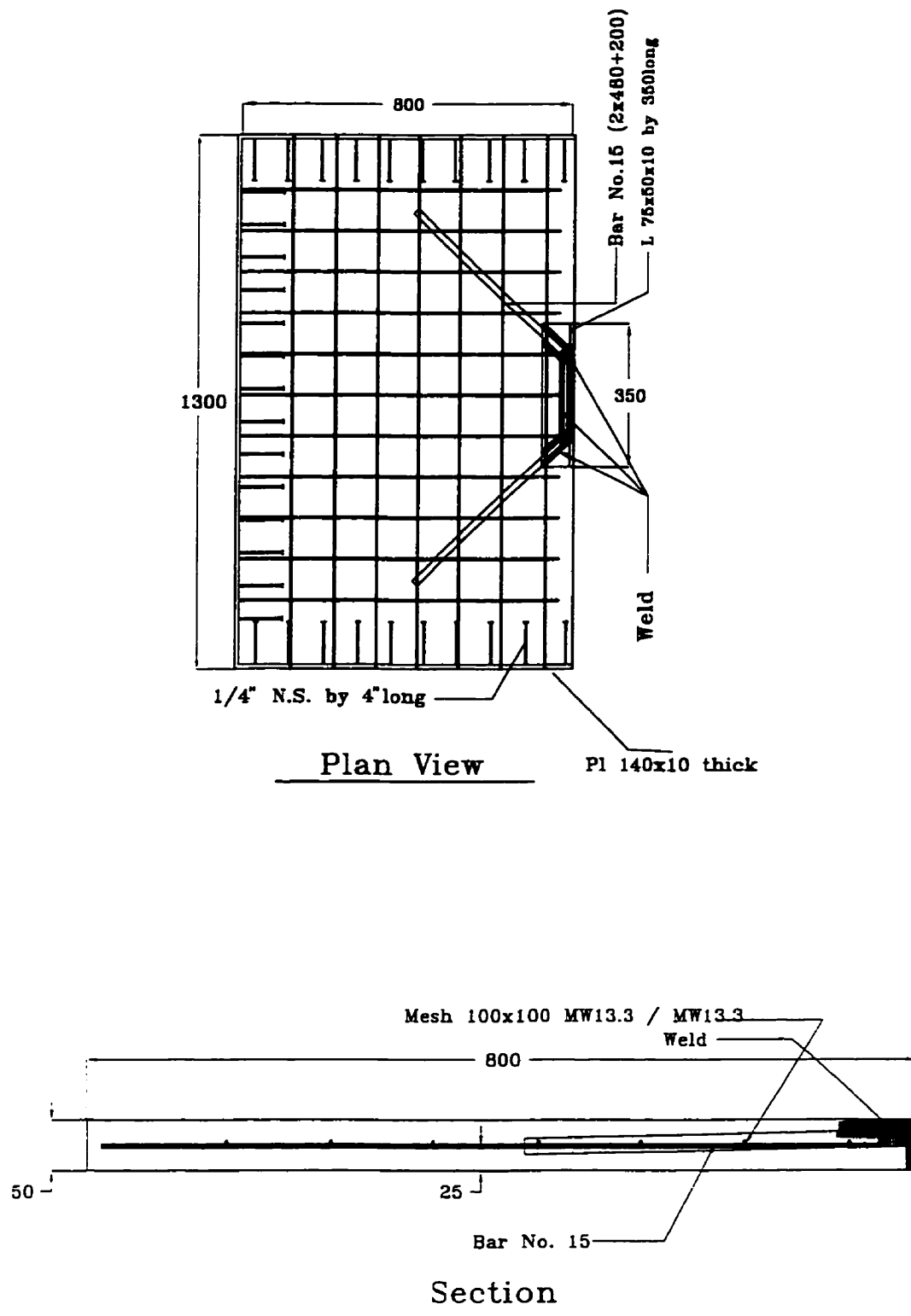


Figure 2.11 Details of Connection Type A

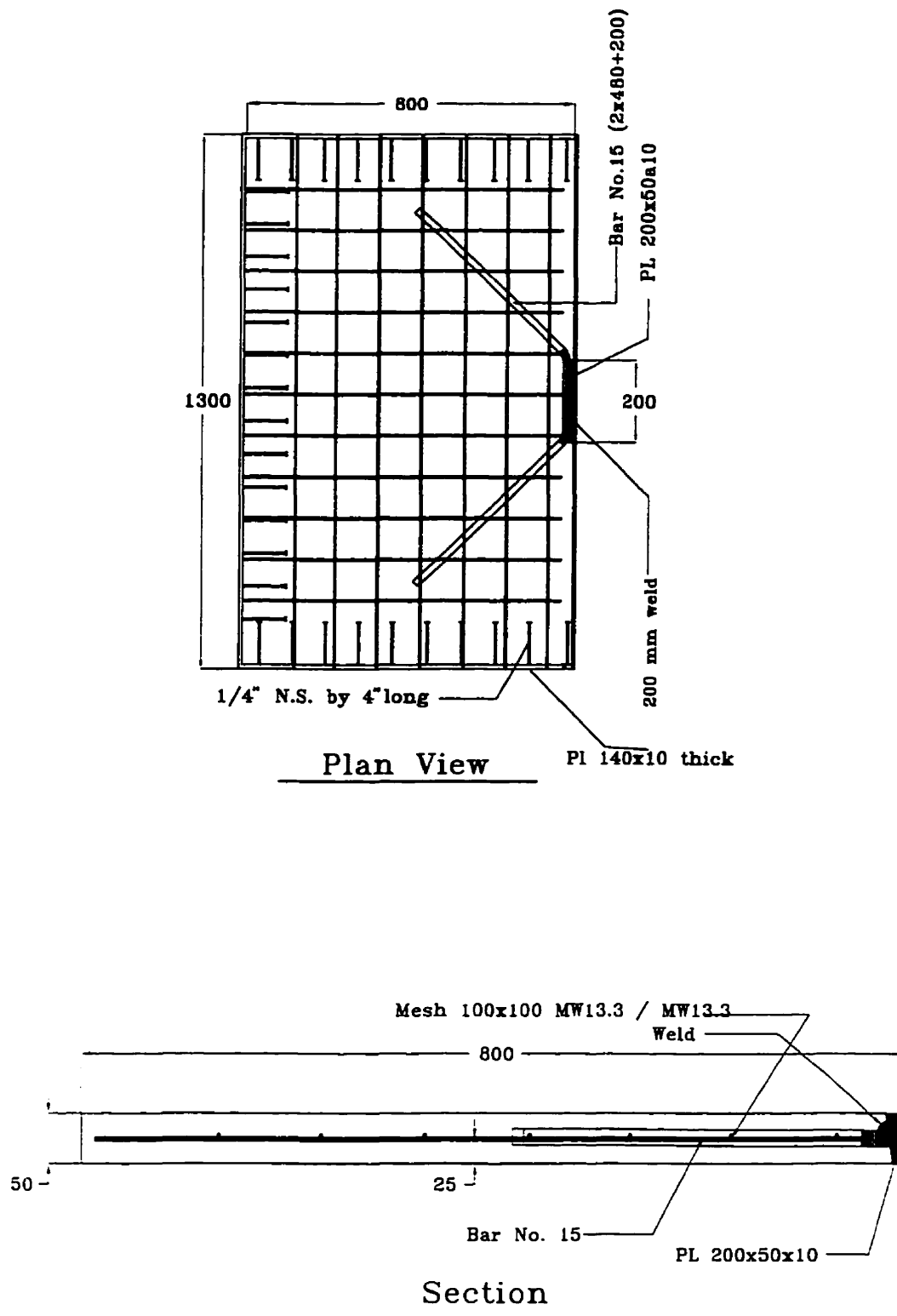


Figure 2.12 Details of Connection Type Ap

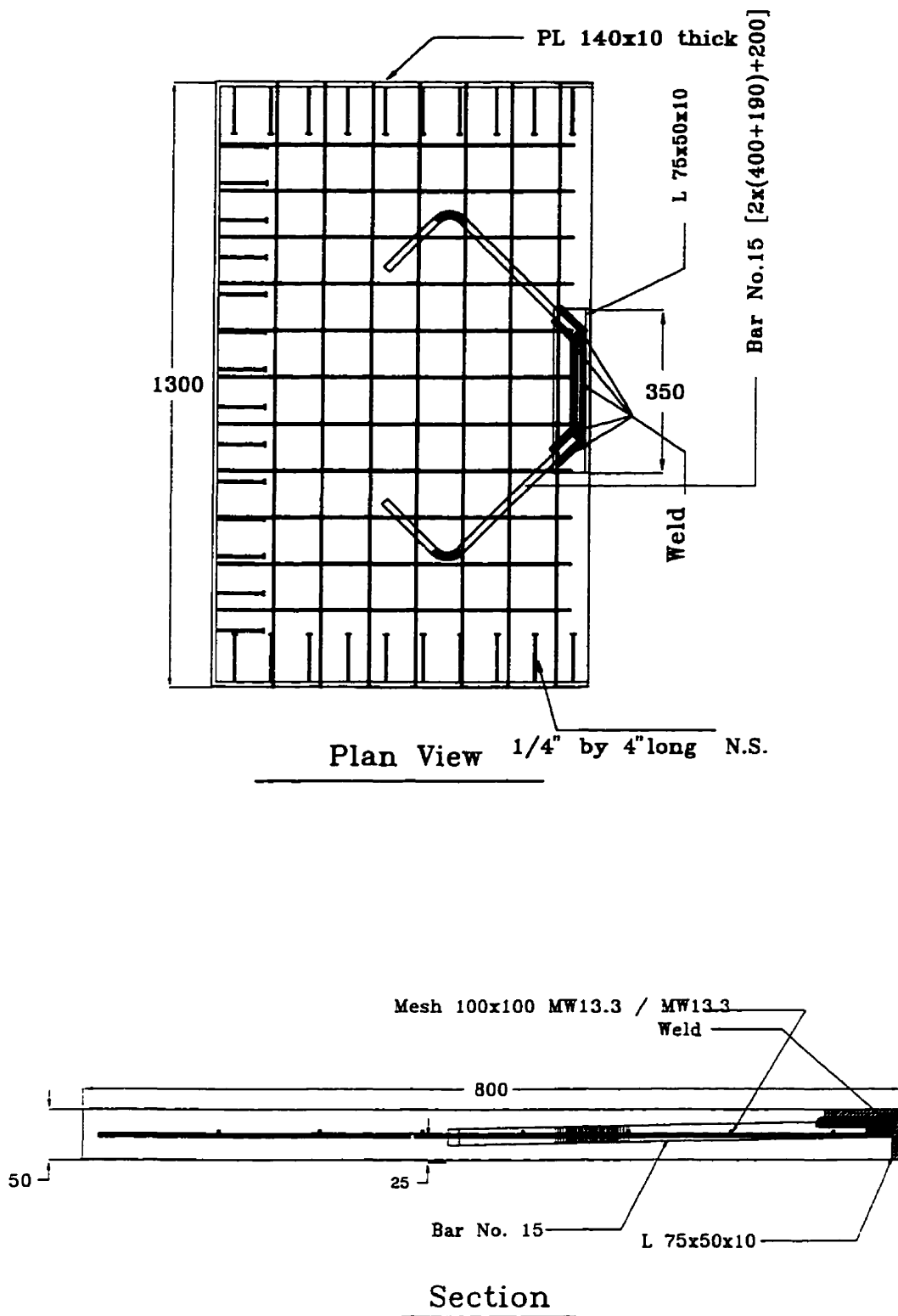


Figure 2.13 Details of Connection Type B

2.4.4 Test Specimens Series C

Figure 2.14 shows the details of the Connection Type C. The Specimens in Series C (Connection Type C) were reinforced with 2 No 15 weldable deformed bars oriented at 90 degrees to the direction of the shear force and welded to the flange of the 50 x 75 x 10 mm thick angle section, 200 mm long. The concrete panel reinforcement was similar to that for Connection Types A and B. The concrete cover thickness of deformed reinforcing bar anchor was the same as in the previous connections. The monotonic ultimate capacity of the connection was calculated using the shear-fraction method (CPCI, 1987). Detailed calculations are presented in Appendix A.

2.4.5 Test Specimens Series D

Specimens in Series D (Connection Type D) were reinforced with three No.15 weldable deformed reinforcing bars with two oriented at 45° to the direction of the shear force and the remaining one at 90°, all the bars were welded to the flange of a 75 mm x 50 mm x 10 mm by 250 mm long steel angle (see Figure 2.15). It was expected that the 90° oriented steel bar at the centre of the connection would help to increase the connection ultimate strength under the combined loading and to secure the connection under twisting and out of plane panel forces caused by the load being eccentric to the angle of the flange. The wire mesh used to reinforced Connection D was identical to that for the other specimens with the steel ratio of $\rho=0.0027$ in both directions. The average clear cover thickness was 20.5 mm. Analysis of Connection is presented in Appendix A.

2.4.6 Test Specimens Series E

The test specimen was reinforced with 2 - 10 mm diameter Nelson studs, 100 mm long fusion welded to a 150 x 50 x 12 mm thick steel plate. The assembly of the headed studs and steel plate was cast in the concrete panel as the centres of the headed studs were coincident with the central plane of the panel. The wire mesh used to reinforce the Connection Type E was similar to that used for the other connections. The connection details are shown in Figure 2.16. Detailed computations for the specimen are presented in Appendix A.

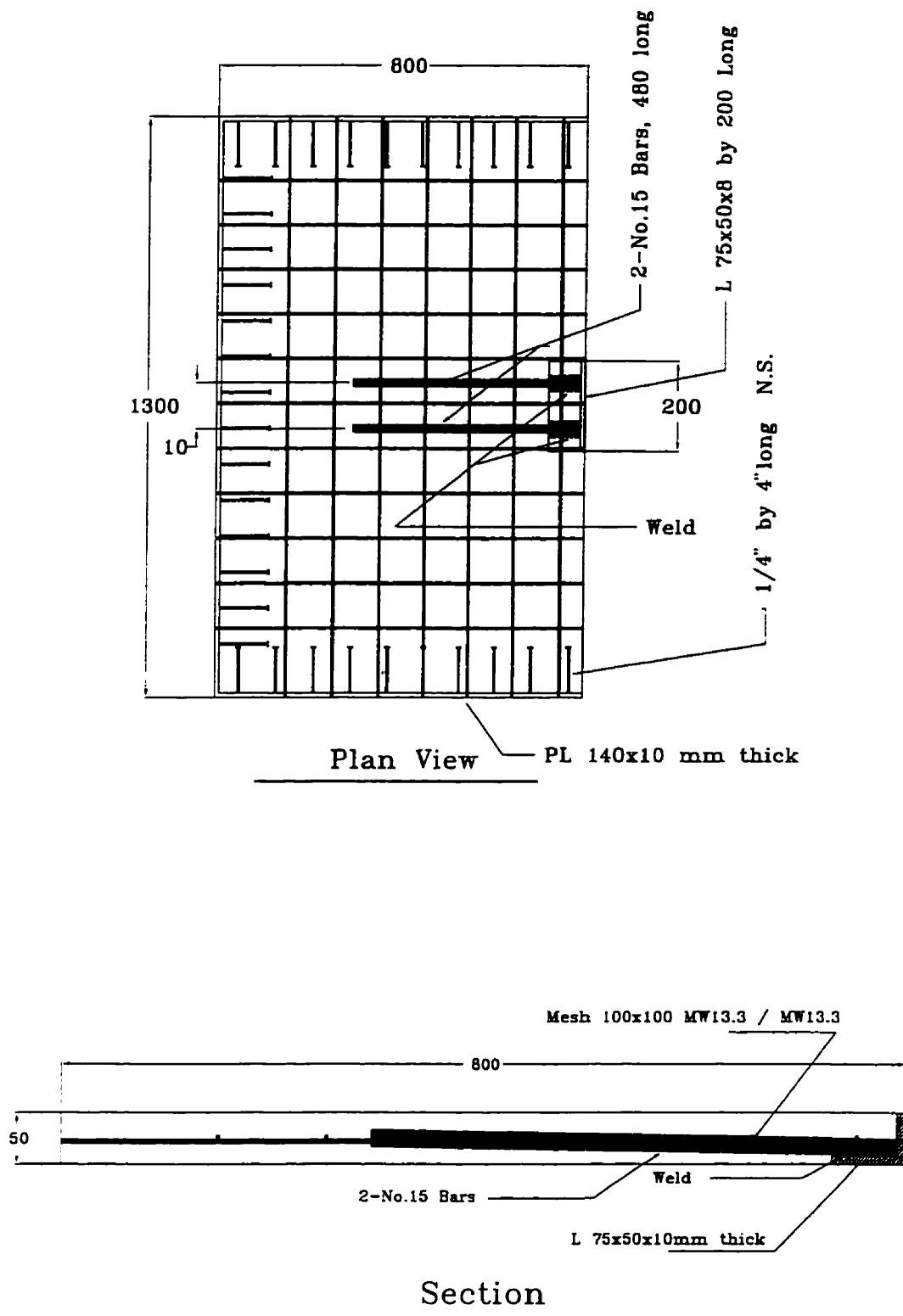
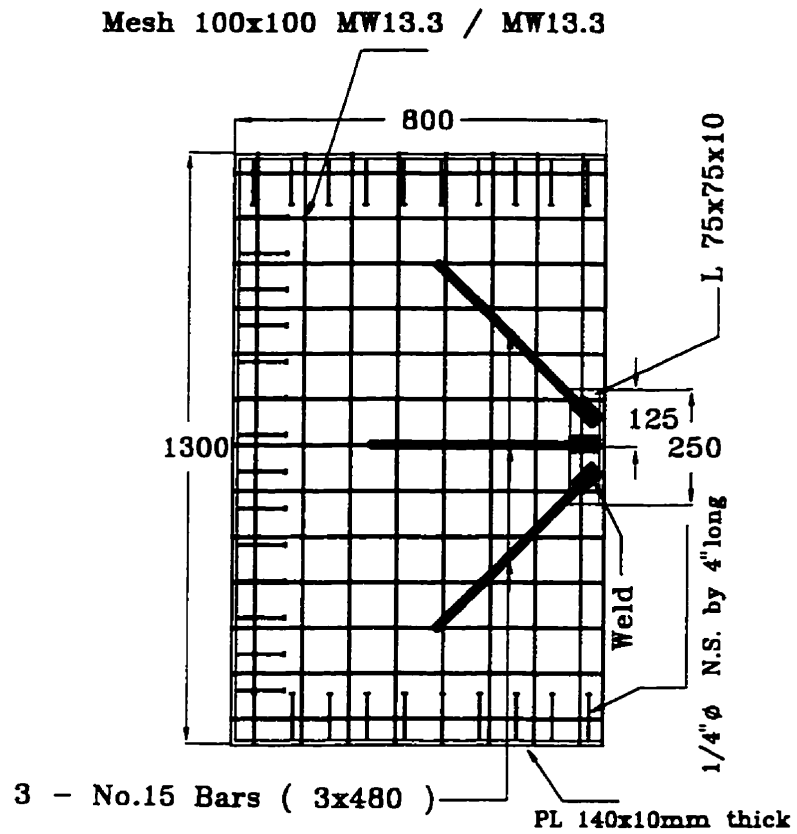
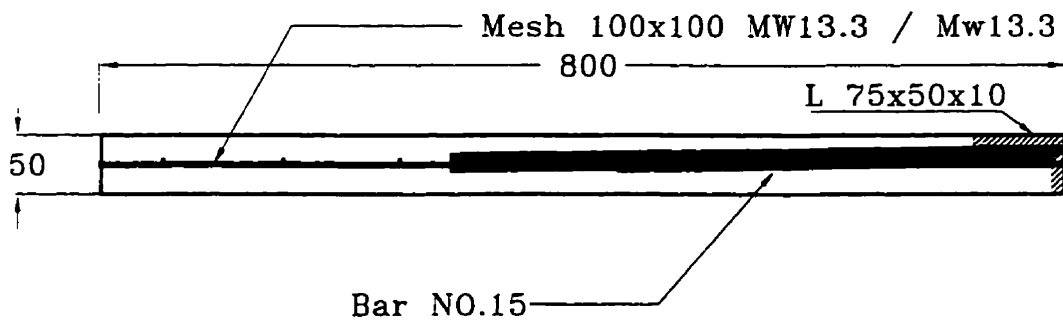


Figure 2.14 Details of Connection Type C



Plan View



Section

Figure 2.15 Details of Connection Type D

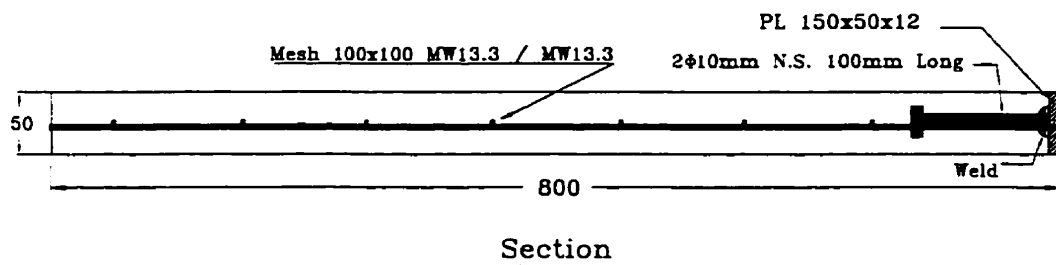
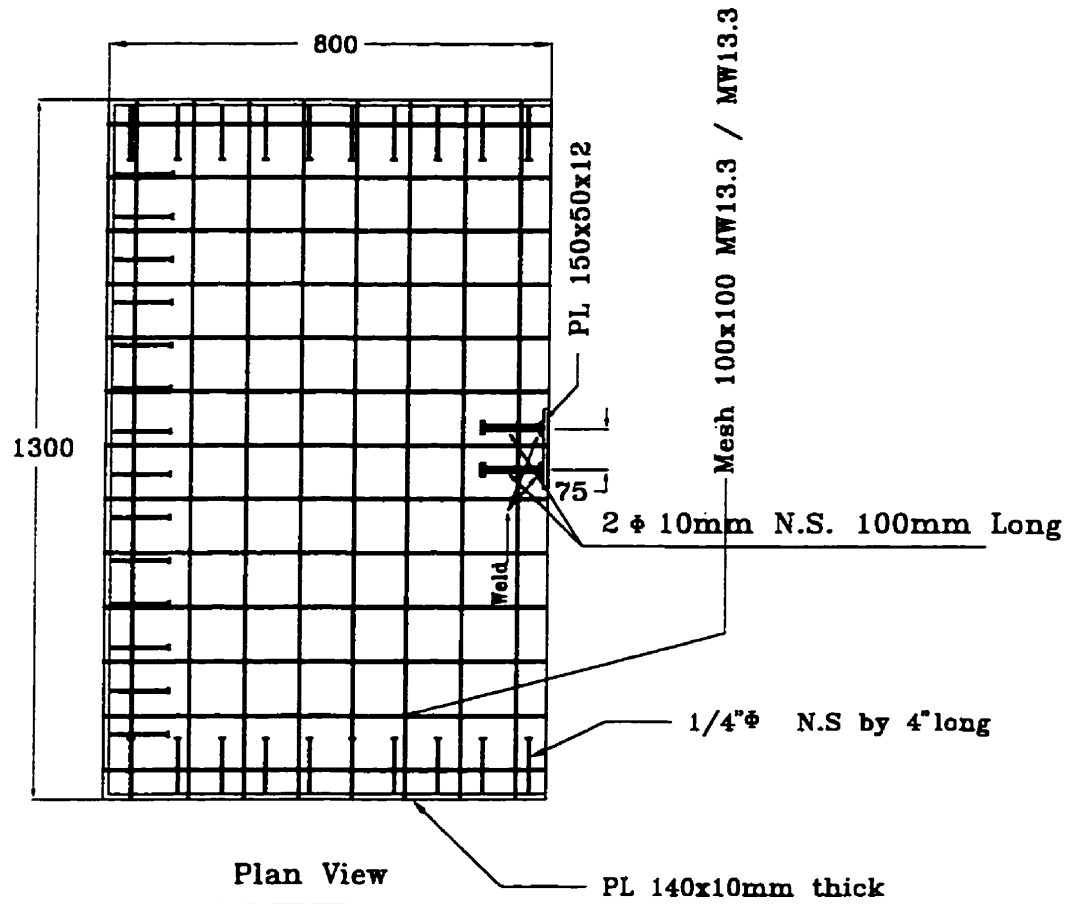


Figure 2.16 Details of Connection Type E

2.5 Construction Sequence

Connections Type A through D

A plate 165 x 225 x 16 mm thick was first cut and six holes were drilled in this plate. This plate was used to apply the in-plane shear and axial loads to the connection for both monotonically increasing and reversed cyclic loadings. The angles and steel plate of appropriate sizes, as described in the previous section, were cut and welded to this plate with a minimum 10 mm thick fillet weld. The deformed reinforcing bar was first cut, bent and then was fillet welded at the centre of the steel angle, or the inclined steel plate. Strain gauges were installed on steel bars as shown in Figures 2.23 to 2.28. The outer frame, was made of steel plate, 149x10 mm thick (Figure 2.21), was placed in the formwork and the assembly of the reinforcing bar and steel plate or angle, and the wire fabric mesh was placed in the plywood formwork so that these bars and the wire mesh were in the middle plane of the concrete panel, see Figures 2.17a to 2.17e. This outer frame was used to connect the specimen boundary to the anchored W-shaped bottom steel beam and the two side beams (Figure 2.20). During the concreting, standard concrete cylinders were cast, and the specimen and the concrete cylinders were cured in wet burlap and then under plastic sheets. The specimens were cast in a horizontal position. The formwork was removed after 2 days, and moist curing was continued for an additional three days.

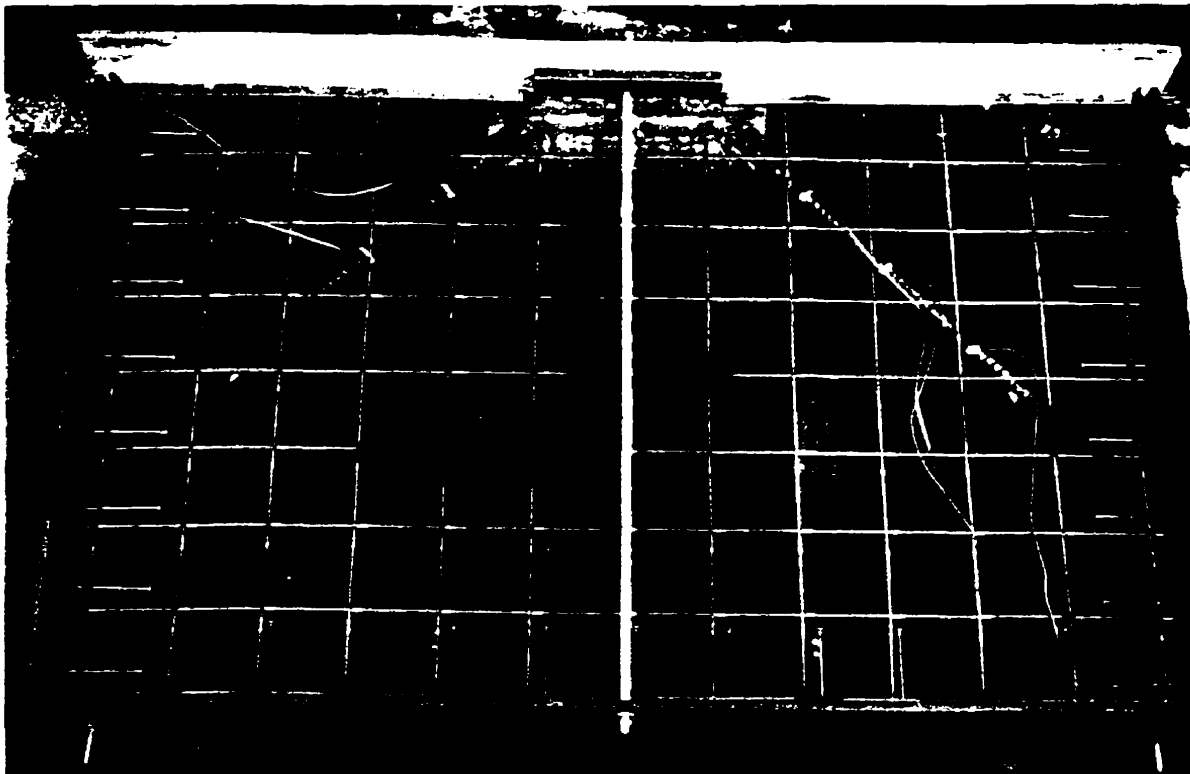
Three small steel channel sections were prepared by drilling a hole at the centre of their webs, and their flanges were welded to the bottom flange of W-shape heavy steel beam (referred to as the anchored steel beam). A number of holes were drilled in the top flange of the bottom W-beam for the connection to the two side W-beams [referred to as the steel beam (W-shape)]. Four strong angles were welded to each side of the W-shaped beams, besides bolting to the flange of the anchored steel beam.

The erection of each test specimen took place in 4 steps. The W- beam was first lifted in the loading frame, welded to the frame bottom beam (Figure 2.19), and was post-tensioned to the strong floor of the Structures Laboratory. The specimen was then hoisted in the holding frame and was positioned vertically on the top flange of the W-beam, and adjusted such that the central plane of the concrete panel was aligned with the web of W-

anchored steel-beam. Then the two W- beams were adjusted to have complete contact with the side of the specimen (outer boundary frame) and bolted. Capping compound was used for prefect contact of the W-beams when these were being bolted to the specimen boundaries. Finally, the base of the specimen was similarly bolted to the anchored bottom steel W-beam.

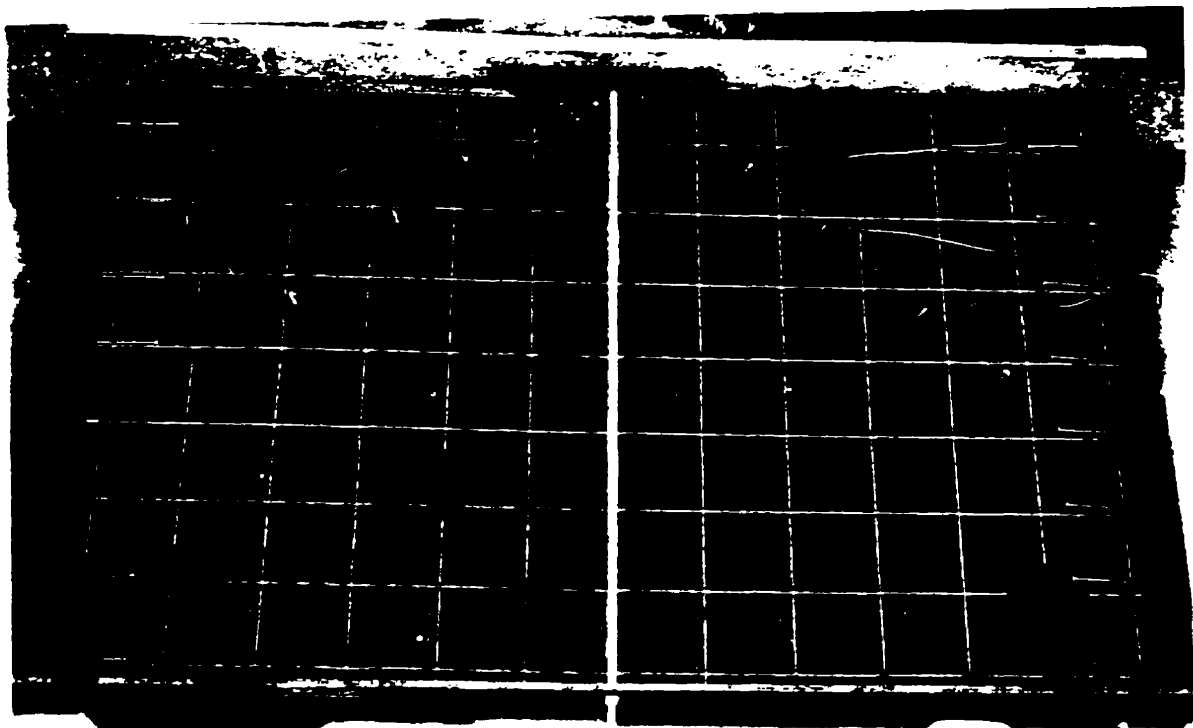
Connection Type E

The construction sequence for Connection Type E was similar to those for previous connections. The headed studs were fusion-welded to a steel plate and the assembly was welded to the plate with holes, followed by the positioning of the whole assembly in the formwork, such that the centre of the heads of studs was coincident with the central plane of the concrete slab (see Figure 2.17f). All other steps and the erection sequence was identical to those for other connections.

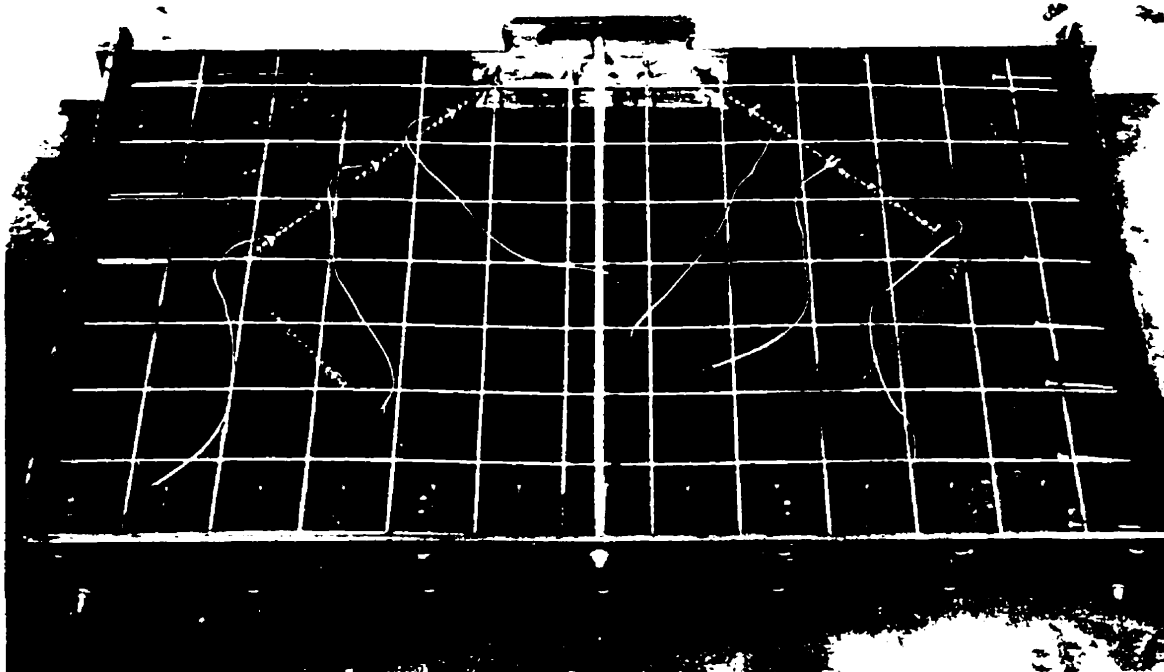


(a) Connection A

Figure 2.17 Formwork for casting test Specimens

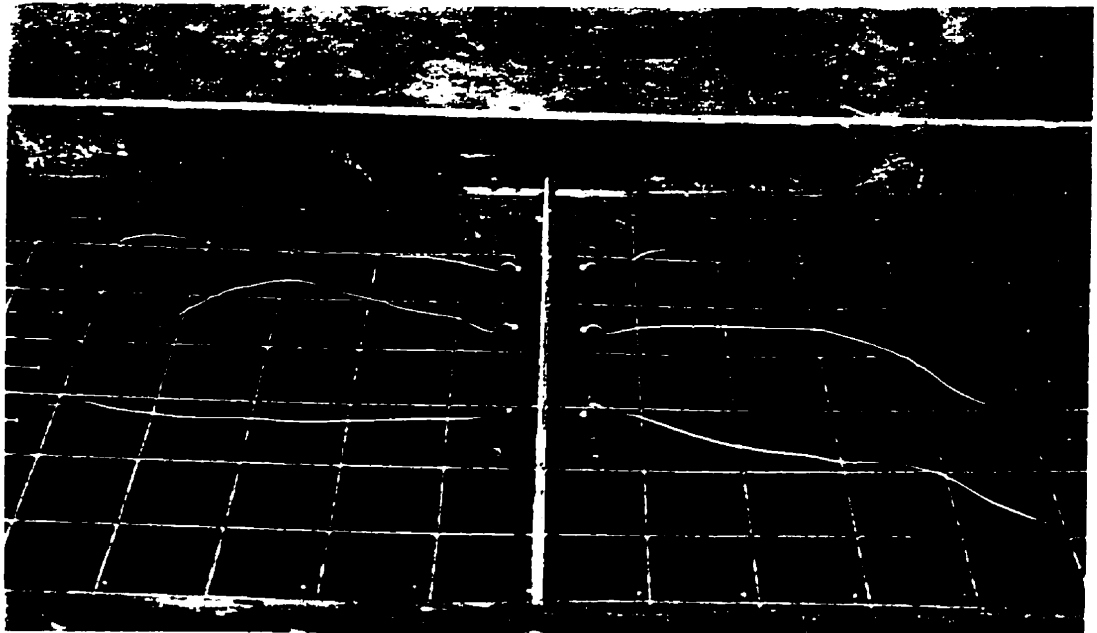


(b) Connection Ap

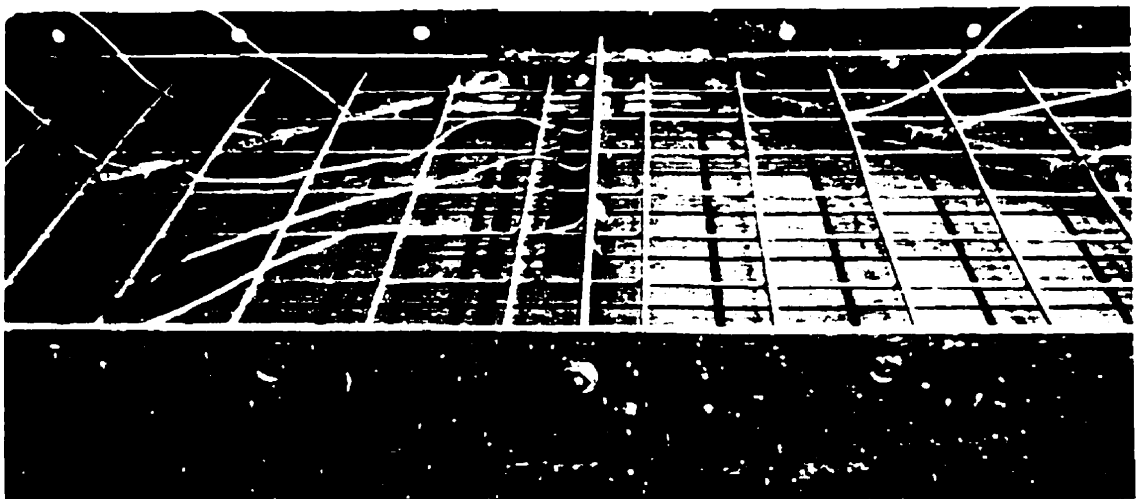


(c) Connection B

Figure 2.17 (Continued) Formwork for casting Specimens

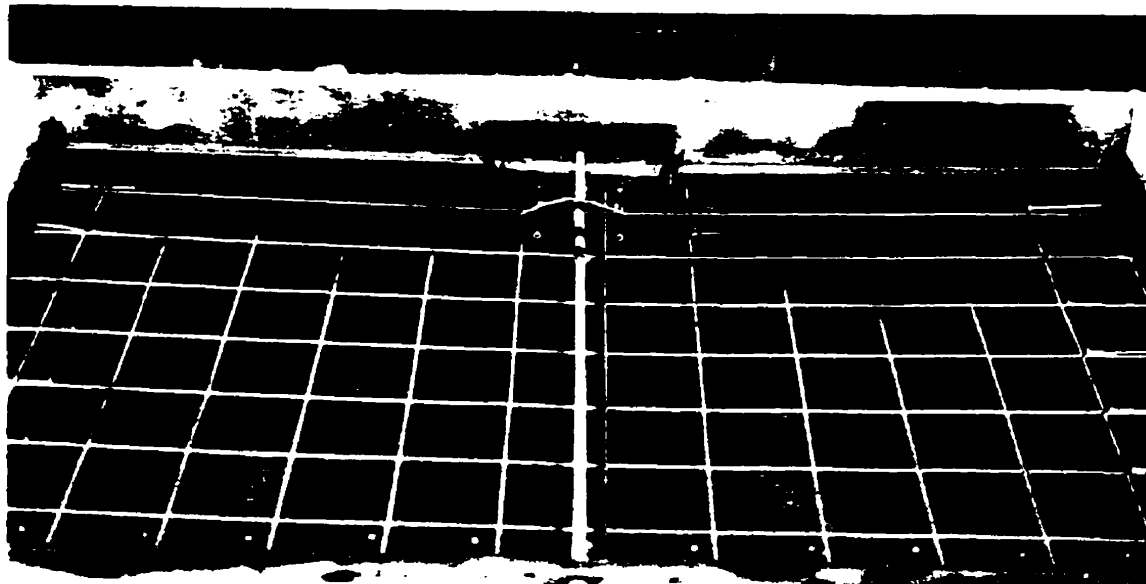


(d) Connection C



(e) Connection D

Figure 2.17 (continued) Formwork for casting Specimens



(f) Connection E

Figure 2.17 (continued) Formwork for casting Specimens

2.6 Material Properties

Concrete

The concrete mix was designed for a 28 - day compressive strength of 35 MPa using Type 30 cement (high early strength), two types of coarse aggregates with maximum sizes of 12.5 mm and 7 mm , and silica (size passing 0.5 mm) sand as fine aggregates and a slump of 75-100 mm. The mix details for the concrete are given in Table 2.1. At least three 150 x 300 mm cylinders were tested for each batch of the concrete used to cast a test specimen, to determine the average compressive strength, f'_c , and the splitting tensile strength, f'_t . The concrete properties are summarized in Table 2.2. A typical stress-strain curve for the concrete is presented in Figure 2.18a. The average tangent modulus, E_c , the compressive strain at the maximum compressive concrete stress,

ϵ'_c , and the tensile strain at the maximum tensile stress, ϵ_t , are also summarized in Table 2.2. The average values were obtained at the time of testing the specimens which varied from an age of 15 to 35 days.

Table 2.1 Mix Design for 35 MPa Concrete

Ingredients	Quantity (kg/m ³)
Cement Type 30	460
Fine Aggregates (silica sand)	830
Coarse Aggregates (12.5 mm)	200
Coarse Aggregates (7 mm)	670
Water	255
Total	2415
Slump	80 ± 10 mm

Table 2.2 Summary of Concrete Properties

Test Specimen	f'_c (MPa)	ϵ'_c (x 10 ⁻³)	E_c (MPa)	f'_t (MPa)	ϵ_t (x 10 ⁻⁴)
Connection A	35	1.71	29752	3.63	1.22
Connection Ap	37	1.80	30767	3.71	1.20
Connection B	36	1.79	32545	3.68	1.13
Connection C	38	1.81	31223	3.72	1.19
Connection D	41	1.75	32676	4.10	1.25
Connection E	40	1.72	33400	3.87	1.16

Steel

The reinforcing steel used for the specimens consisted of No. 15 weldable bars with a minimum specified yield strength of 400 MPa. The weldable steel complied with the requirements of the CSA G30.16 -M (1994). Samples from deformed reinforcing bars were tested using the standard tension test to establish their properties. A 50 mm extensometer was used to determine the steel strains. Table 2.3 presents a summary of the steel properties. A typical stress-strain curve is shown in Figure 2.18b.

The ultimate strength for headed studs was assumed to be 400 MPa for design and analysis.

Table 2.3 Summary of Reinforcing Steel Properties

Property	No. 15
Yield Stress, f_y (MPa)	465
Young's Modulus, E_s (GPa)	210
Yield Strain, ϵ_y (%)	0.220
Maximum Stress, f_{max} (MPa)	632
Ultimate Strain, ϵ_{ult} (%)	12.02
Strain-hardening Modulus, E'_s (MPa)	1860

E'_s is used in NONLACS program

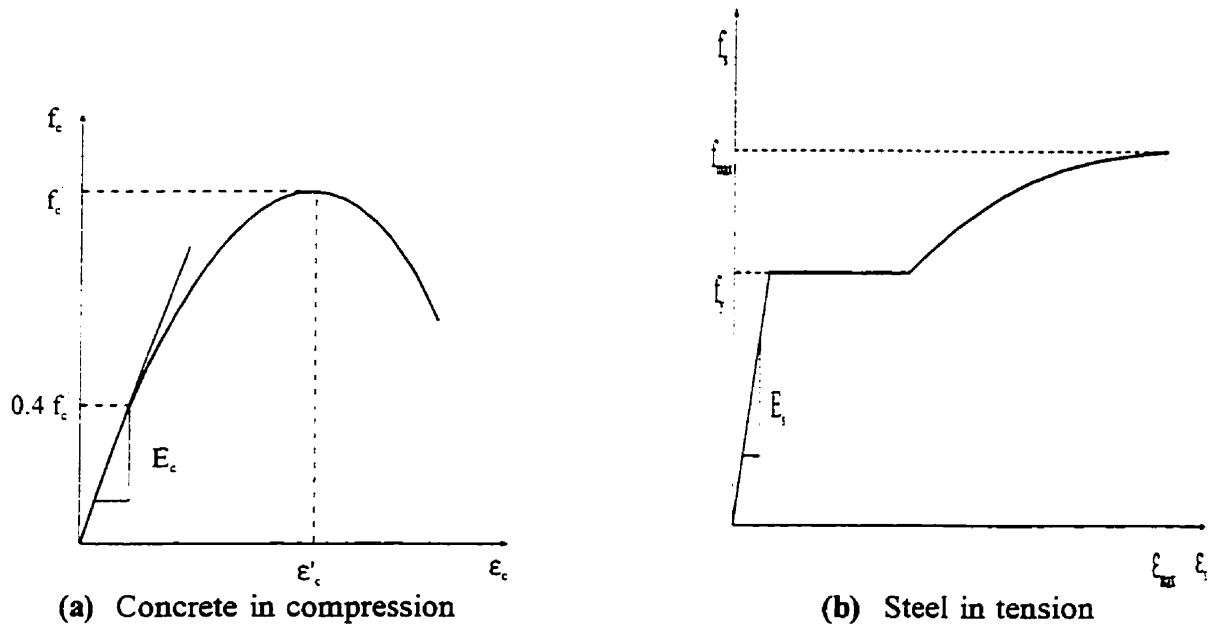


Figure 2.18 Stress-strain curves for concrete and steel

2.7 Test Set-Up

Figure 2.19 presents the details of the test set-up and the loading frame. Reversed cyclic shear and axial forces were applied to the test specimen by four sets of hydraulic jacks. Two sets of jacks were used to apply the shear and axial (tension force) forces in the positive direction whereas the other two sets were used in the negative direction (shear and compressive axial forces). The loading frame facilitated application of in-plane axial and lateral loads (axial and shear forces) and consisted of braced back-to-back channel beams post-tensioned (clamped) to the strong floor of the Structures Laboratory. The lateral displacement of the loading frame was measured during the test and found to be negligible.

As shown in Figures 2.20 and 2.21, the reversed cyclic shear forces were applied to the connection by high strength threaded rods, pin-connected to a hollow steel section, which was welded to a steel plate to transfer pure shear to the connection. The axial tensile and compressive forces (reversed cyclic axial loads) were applied to the connection by high strength rod, pin-connected to a hollow steel section, and a vertical jack placed between the hollow steel section and the top beam of the reaction frame (Figures 2.19 and 2.21), respectively.

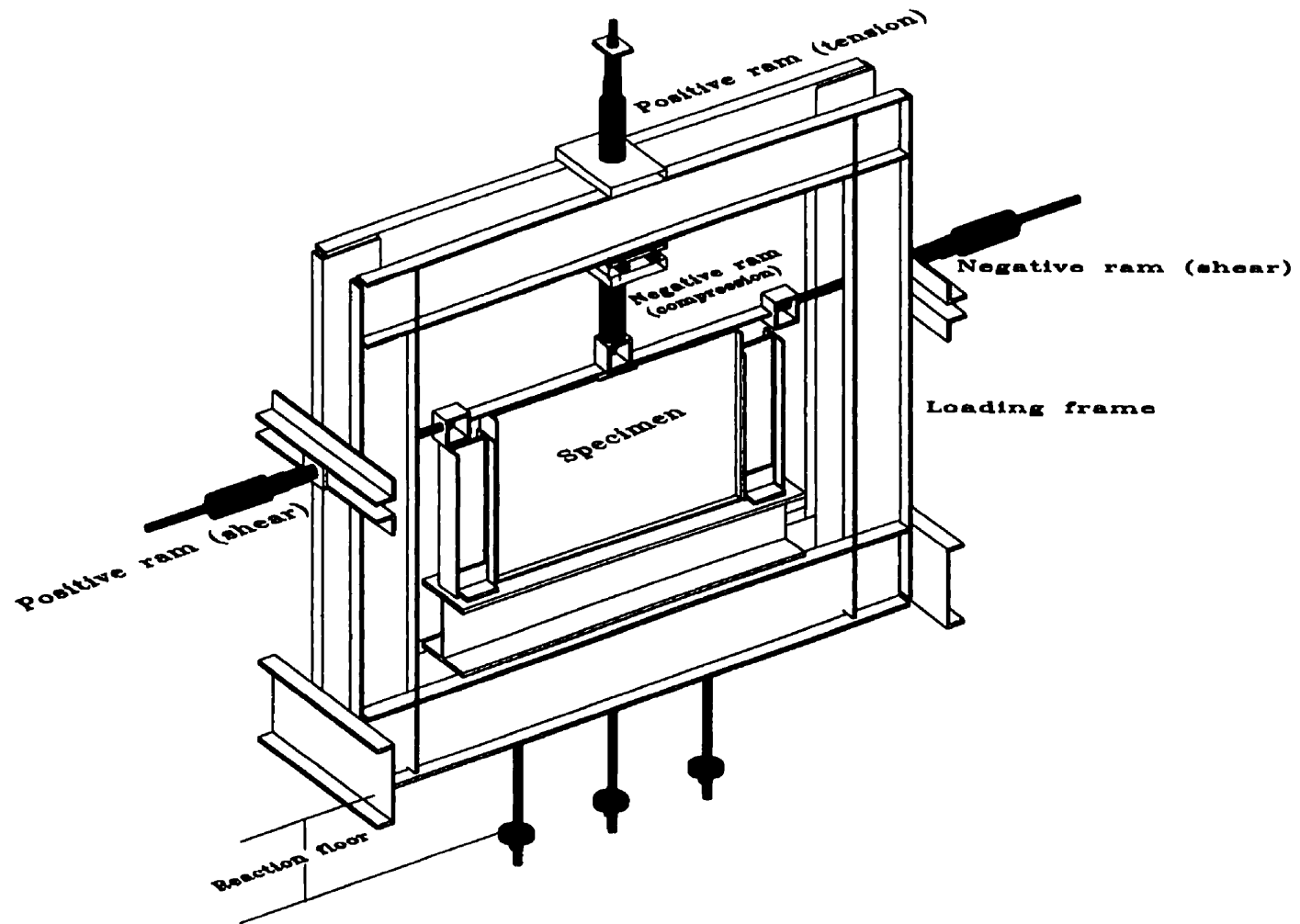
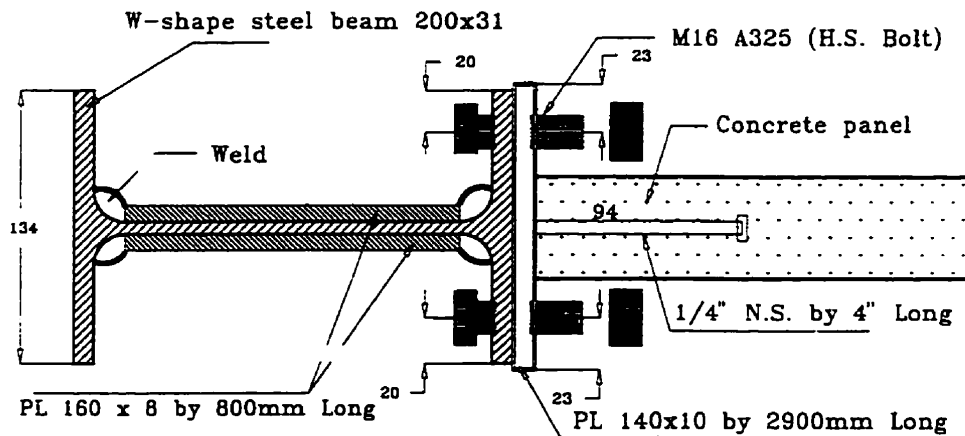
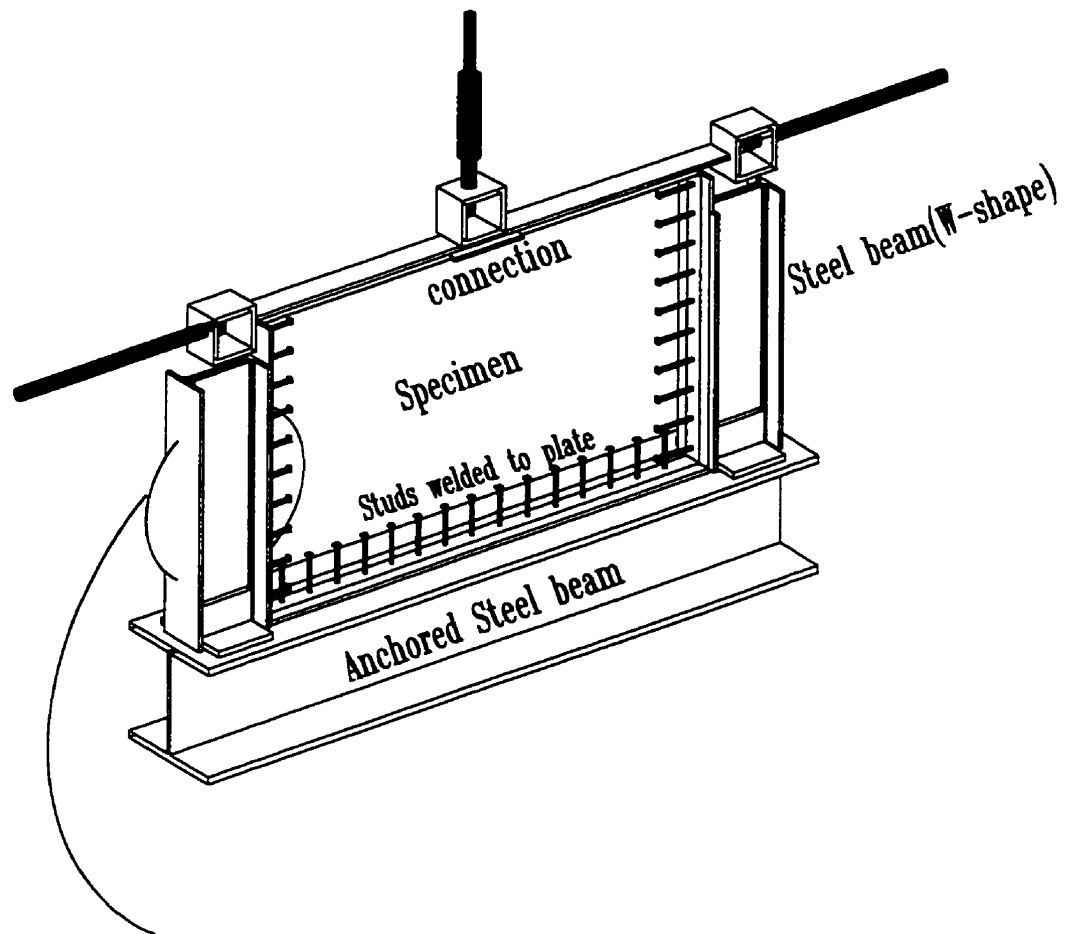


Figure 2.19 Test Set-Up - Reaction frame



Plan view of boundary of specimen

Figure 2.20 Test Set-up - Test Specimen boundary condition and application of loads

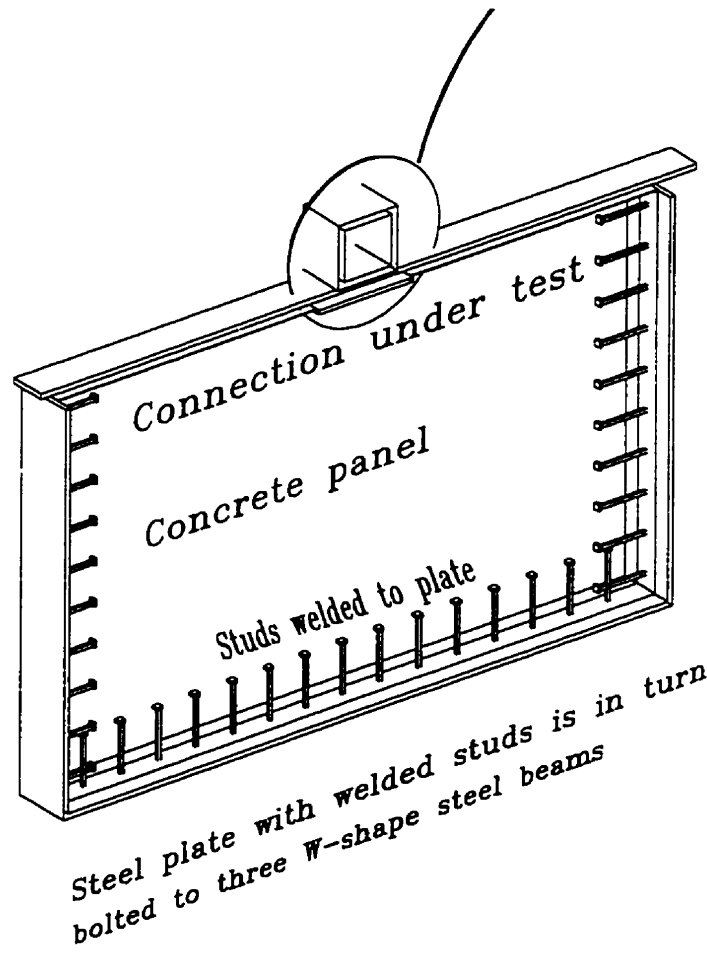
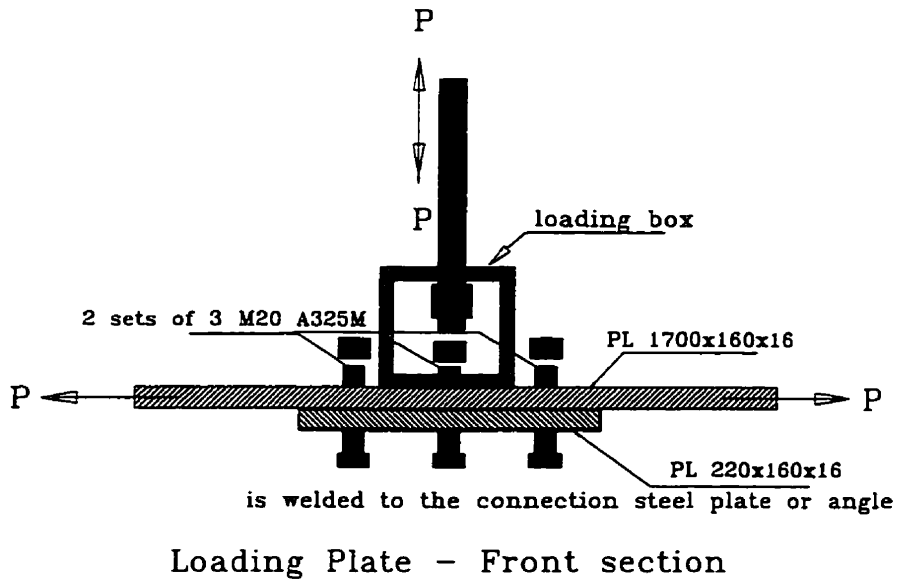


Figure 2.21 Test Set-Up - Specimen details (top)

2.8 Loading History

The specimens were subjected to pure cyclic shear, pure cyclic axial loads and different combinations of cyclic shear and axial loads. The two sets of general loading history presented in Figure 2.22 were employed for application of combined in-plane reversed cyclic shear and axial load during the testing of the connection specimens. The first six cycles were "load controlled" and consisted of three cycles in the elastic range of response, followed by three cycles beyond the cracking load of the test specimens.

The deflection at yielding, Δ_y , was defined as the horizontal displacement at which the steel at Location 1 or 4 yielded, accompanied by a reduction in stiffness which was noted from the load-deflection curves. The subsequent load cycles, beyond Δ_y , were continued using "displacement-control". Three displacement cycles at $\pm 1\Delta_y$ were imposed followed by three displacement cycles each at $\pm 2\Delta_y$, $\pm 3\Delta_y$, $\pm 4\Delta_y$, etc. The exact loading history and level of loading for each specimen, along with the hysteresis response are presented in Chapters 3, 4, 5.

For the combined loading cases, the axial force was applied with a delay of a quarter cycle after application of the shear force. It was not always possible to control both the shear and the axial loads to maintain a constant T/V ratio, where T is axial force and V is shear force. However, the T/V ratio during the loads controlled part of the experimental was maintained as close to the planned T/V ratio as possible. Again, for the same reason, where the experimental procedure was changed from load control to displacement control, the calculated ductility ratio was based on the resultant displacement, calculated as the vector sum of the "horizontal" displacement (in the direction of the shear loading) and "axial" displacement (in the direction normal to the shear loading and along the direction in which the axial tension and compression were applied).

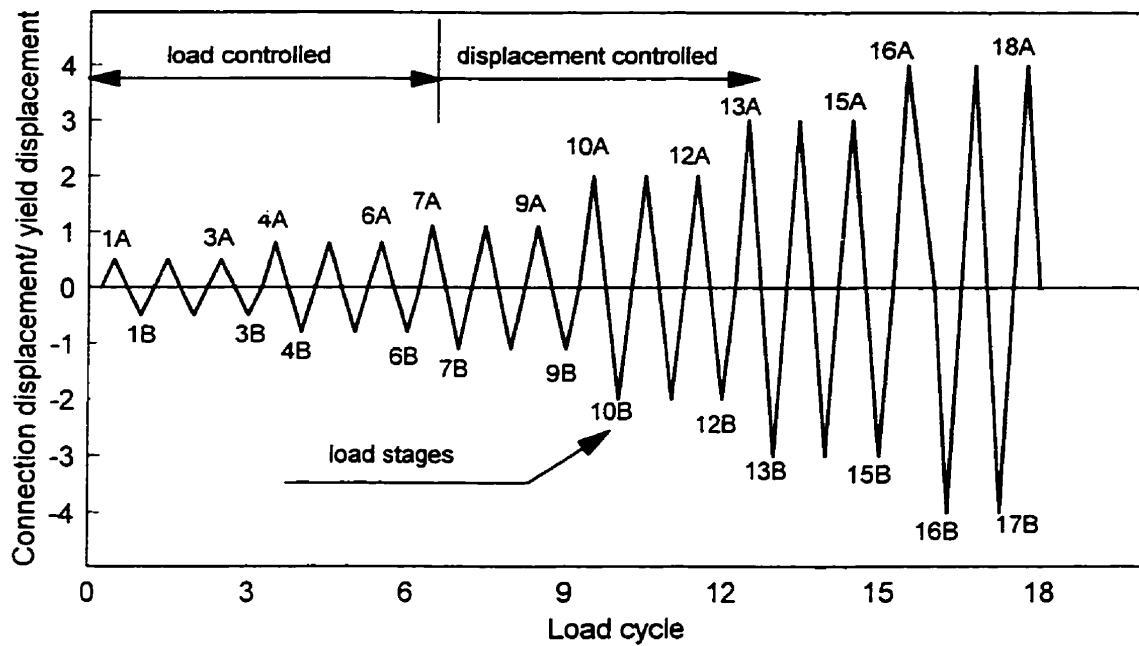


Figure 2.22 Typical loading history

2.9 Instrumentation

Four 330 kN load cells were used to measure the in-plane shear and axial forces applied to the specimen in the positive and the negative directions. Six or nine electrical resistance strain gauges were used to measure the strains along the length of the outermost No.15 anchor bar in each specimen, as shown in Figures 2.23 through 2.28.

Linear variable differential transformers (LVDT's) were used to measure horizontal and vertical deformations of the connection and horizontal deformation of the concrete specimen (see Figures 2.23 to 2.28). A data acquisition system (Doric 245) was used to monitor the steel strain gauges, the LVDT's and the load cells. A reduction system which converted the voltages into loads, displacements and strains was used.

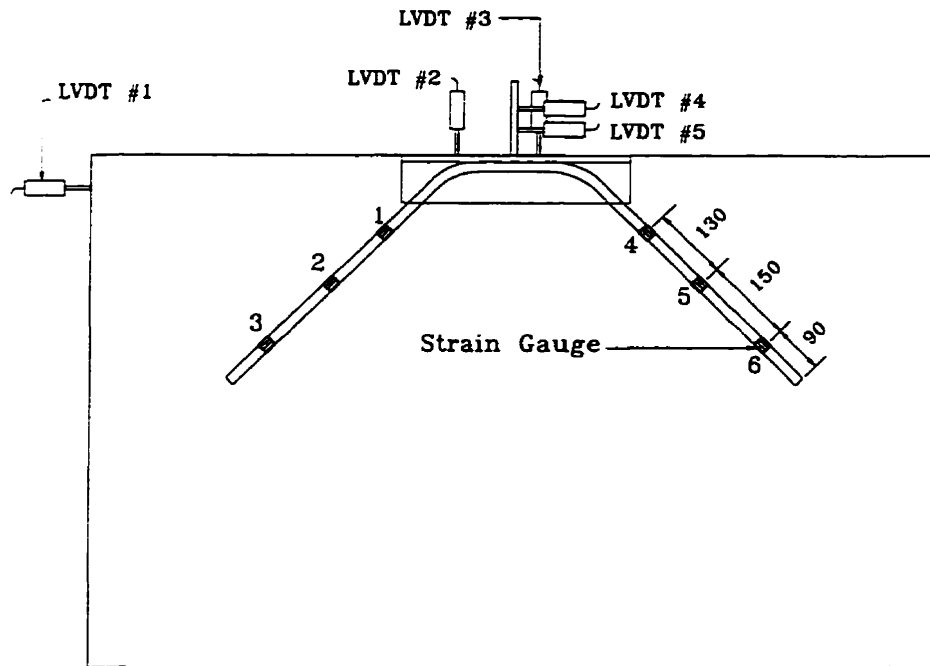


Figure 2.23 Instrumentation for Connection Type A

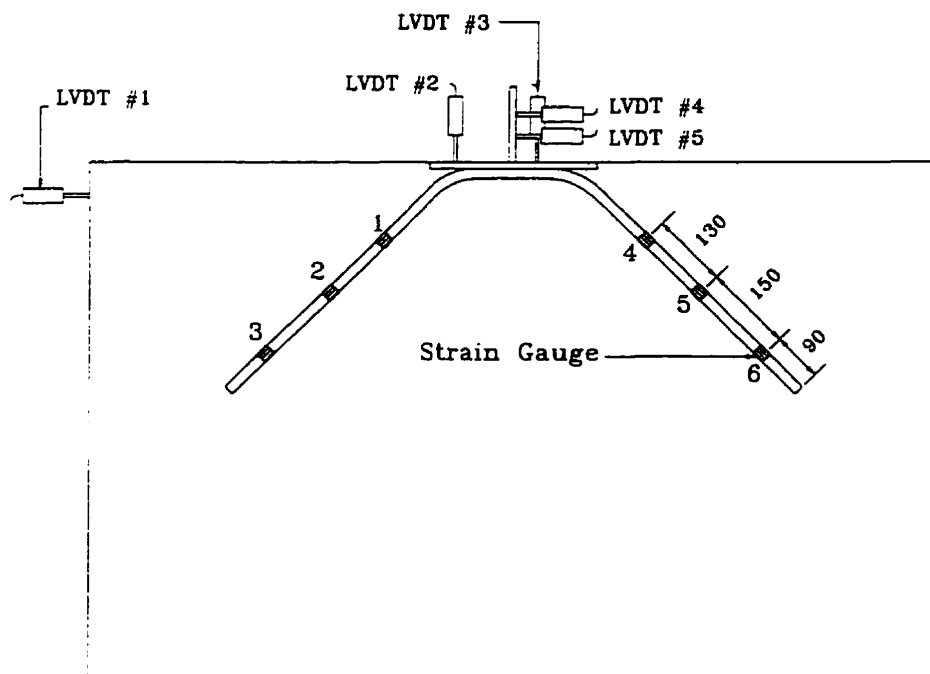


Figure 2.24 Instrumentation for Connection Type Ap

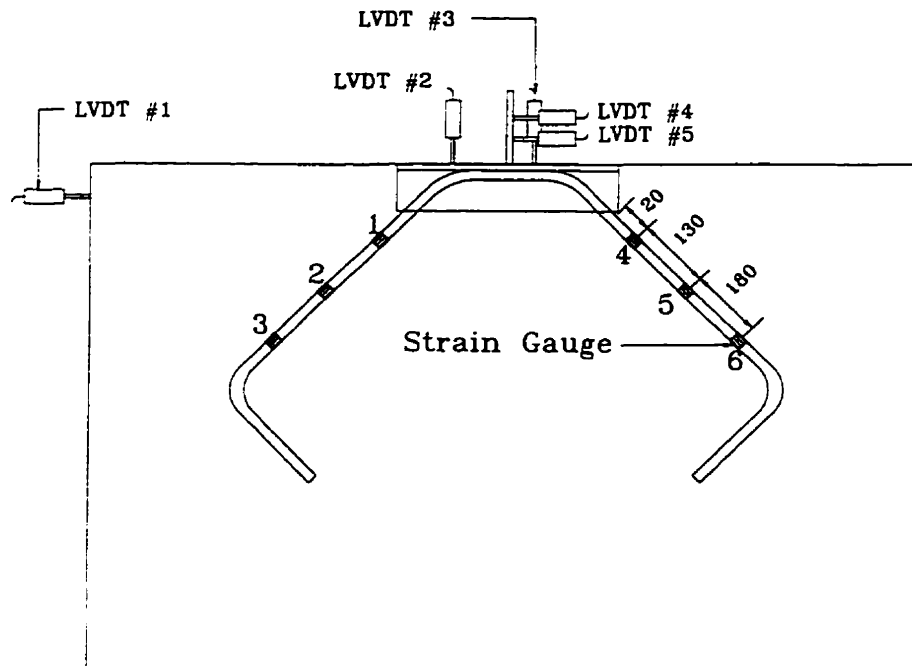


Figure 2.25 Instrumentation for Connection Type B

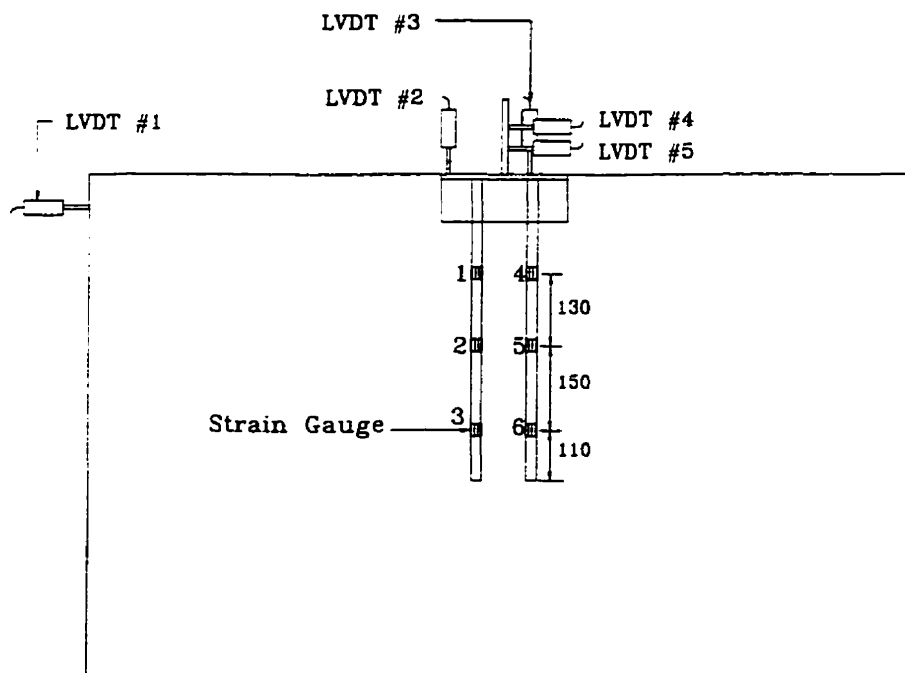


Figure 2.26 Instrumentation for Connection Type C

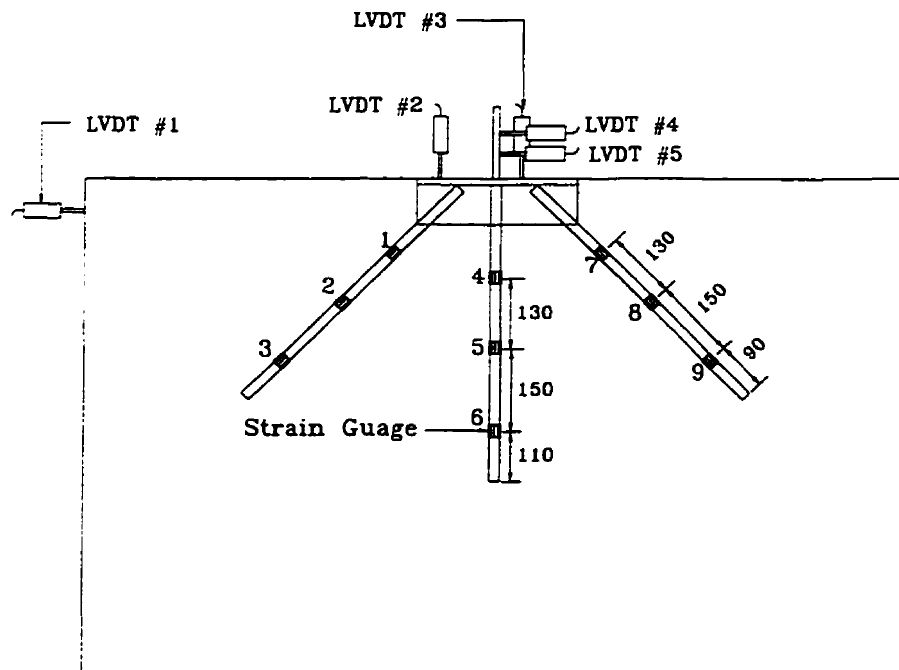


Figure 2.27 Instrumentation for Connection Type D

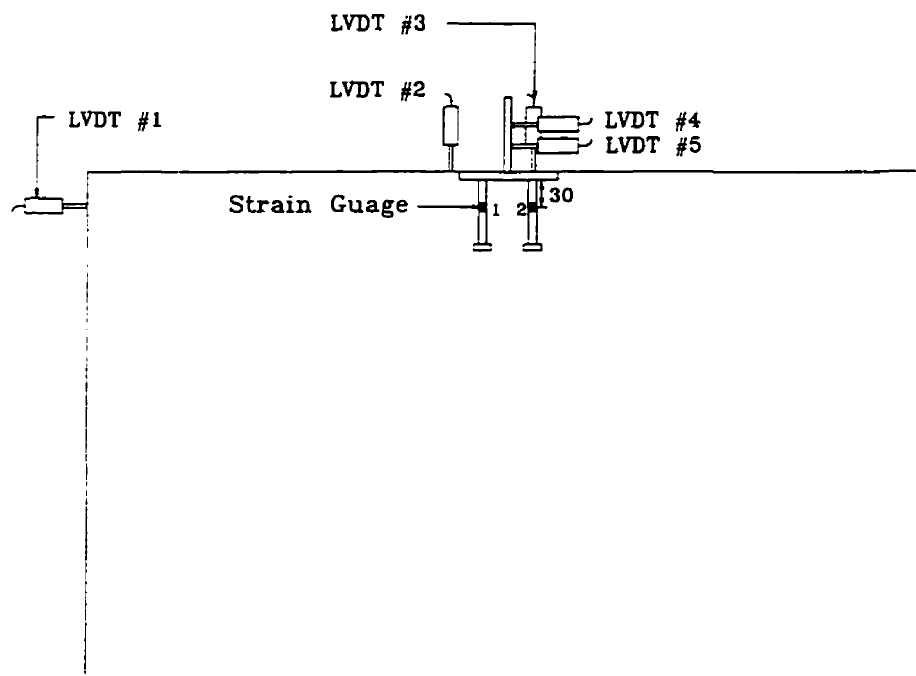


Figure 2.28 Instrumentation for Connection Type E

Chapter 3

Experimental Results - Series A and B

3.1 Introduction

The complete responses of two sets of six identical connections (Series A and B), tested to destruction under monotonically increasing and reversed cyclic loads along with the test of Connection Type Ap subjected to reversed cyclic shear loads (Specimen ApS) are described in this chapter. The hysteresis response of the specimens under monotonically increasing shear loads and combined reversed cyclic shear and axial (tension-compression) loads, shear force-axial force interaction diagrams, total specimen displacements, strains at six locations on the reinforcing bar anchor were measured.

To investigate the cyclic response of the connections subjected to combined axial and shear forces, three specimens of each series were subjected to cyclic combined shear and axial forces with three different values of T/V ratio, where T is axial force and V is shear force. The axial forces were applied a quarter cycle delay after application of the shear forces. It was not always possible to control both the shear and the axial loads to maintain a constant T/V ratio. However, the T/V ratio during the load controlled part of the experiment was maintained as close to the planned T/V ratio as was possible. Again, for the same reason, where the experimental procedure was changed from load control to displacement control, the calculated applied displacement ductility ratio was based on the resultant displacement, calculated as the vector sum of the "horizontal" displacement (in the direction of the shear loading) and the "axial" displacement (in the direction normal to the shear loading and along the direction in which axial tension and compression were applied).

For each of the connection specimens, the following figures are presented :

1. Monotonically increasing applied loads vs. horizontal displacement(monotonic shear response).
2. Cyclic applied load vs. horizontal displacement response (cyclic shear response).

3. Cyclic applied load vs. vertical displacement response (cyclic axial response).
4. Applied load vs. measured strain in the reinforcing bar anchors studs.
5. Measured strain vs. distance along the embedded anchors at ultimate load.
6. Photographs of the specimen at different load stages.
7. Shear force-axial force interaction diagrams for each series.

3.2 Test Series A: Connection with bent steel bar anchor welded to steel angle

Series A consisted of six identical test specimens (Specimens AMS, AS, AST1, AST2 and AST3), as shown in Figures 2.1(a) and 2.11. The specimens in Series A were reinforced with a No. 15 deformed bar, bent at 45° welded to a 10 mm thick steel angle and a uniform wire fabric mesh, 100 x 100 MW13.3 / MW13.3 (Figure 2.11). It was designed and detailed with special consideration for development length under low cycle loading (ACI, 1992).

Determination of the ultimate strength under monotonically increasing load (referred to as "monotonic" load) is based on the anchorage length for reversed cyclic loading, and the equations which in turn are dependent on the concrete cover thickness. All calculations were performed considering equilibrium of the joint using the truss analogy. The shear resistance of the concrete in front of the steel angle or plate was considered in the calculations of the shear strength of the connections; the wire fabric mesh was not included in the calculation.

3.2.1 Specimen AMS

Figure 3.1 shows the shear force-horizontal displacement curve for Specimen AMS. The load versus measured strains along the length of anchor are presented in Figure 3.2. The profile of measured strain vs. distance along the embedded anchors prior to failure is shown in Figure 3.3. Figures 3.4(a) and 3.4(b) show photographs of the connection at the yield level and after failure, respectively. The following important behavioural aspects were noted during testing and analysis of the experimental results.

Stage 1, 85 kN The first splitting cracks appeared along the tension reinforcing bar, along with cracking along the compression leg of the reinforcing bar which was due to the dowel action resulting in a sudden increase in the displacement and change in the slope of the shear- horizontal displacement curve.

Stage 2, 140 kN As the load was increased further, the splitting cracks extended along the reinforcing bar, and diagonal cracks appeared perpendicular to the splitting cracks. The maximum lateral displacement and the maximum crack width at this load stage were 1.61 mm and 0.33 mm, respectively.

Stage 3, 187 kN The ultimate capacity (187 kN) was achieved (step 3 in Figure 3.1) and then the load dropped suddenly to 110 kN (step 4) due to the crushing and spalling of the concrete in front of the angle and splitting of the concrete along the tension leg of the reinforcing bar as shown in Figure 3.4(a). At this stage, the load was carried mostly by the tension leg and that caused the strain at Location 2 (Figures 3.2a and 3.2c) to increase to 2756×10^{-6} and yielding of the reinforcing bar. The maximum lateral displacement at this load stage was 3.0 mm (Δ_y). Figure 3.2 shows the measured strains along the tensile leg confirming the yielding of reinforcing bar at Location 2. The maximum measured crack width at this load stage was 0.8 mm along a diagonal crack perpendicular to the reinforcement. As the load was increased further, the connection could not carry any more load and the lateral displacement of the connection increased significantly, as can be seen from Figure 3.1. The connection failed at a load of 109.6 kN. The maximum displacement was 5.0 mm, giving a displacement ductility ratio of $\mu = 5.0/3.0 = 1.7$.

The variation of strain along the reinforcing bar prior to failure of the connection is presented in Figure 3.3, which indicates that about one-third of the tension reinforcing bar had yielded. The ultimate load for connection AMS was 187 kN, after which the load dropped suddenly to 110 kN. The reinforcement yielded at this load. A displacement ductility of 1.7 was achieved for this connection. The connection failed at a load of 109.5 kN due to the crushing of the concrete in front of the angle, with spalling of the concrete along the compression leg, splitting and spalling of the concrete along the tension leg, and

due to the out of plane rotation of the angle. Figure 3.4(b) shows the photograph of the connection after testing, which explains the failure mode of the specimen.

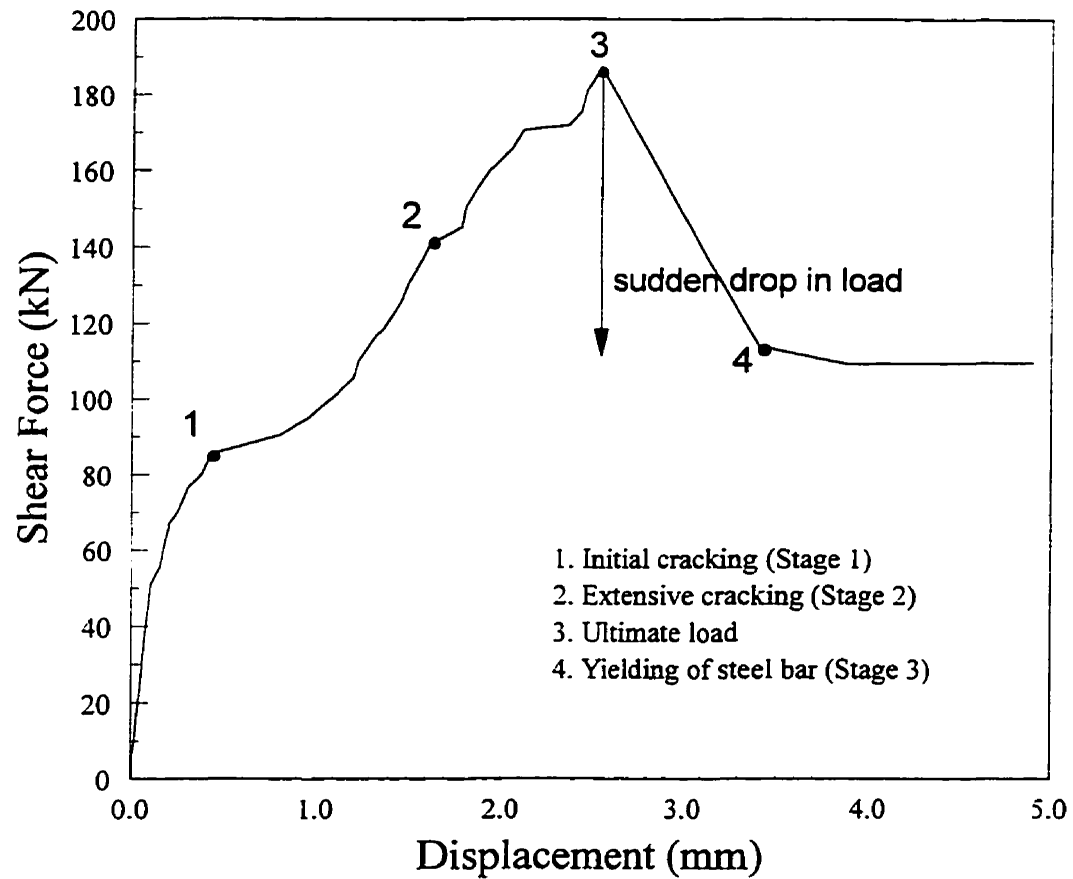
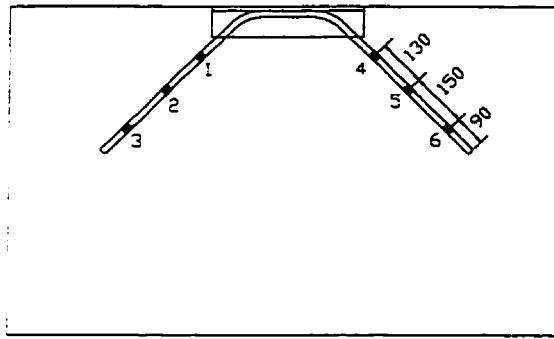
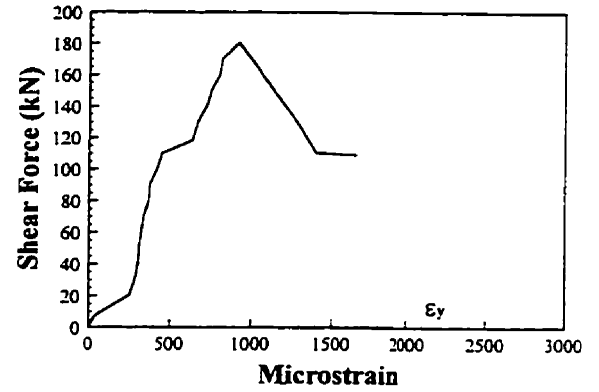


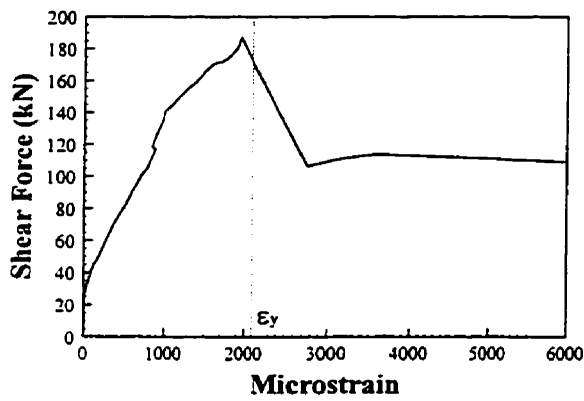
Figure 3.1 Shear force vs. displacement curve for Specimen AMS



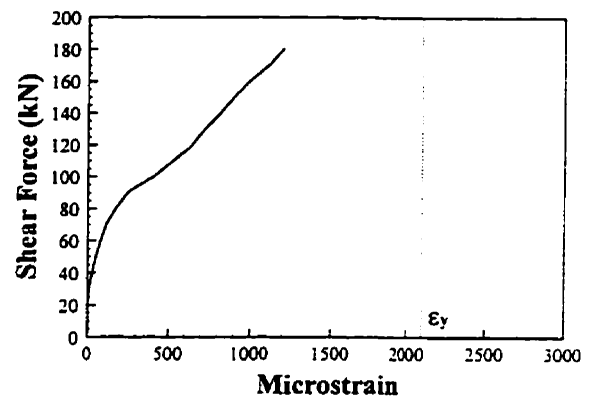
(a) Location of strain gauges



(b) Strain at location 1



(c) Strain at Location 2



(d) Strain at Location 3

Figure 3.2 Measured strains along the length of anchor for Specimen AMS

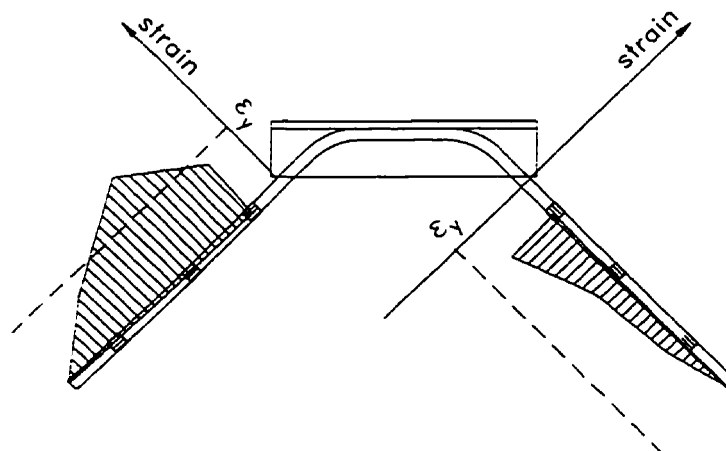
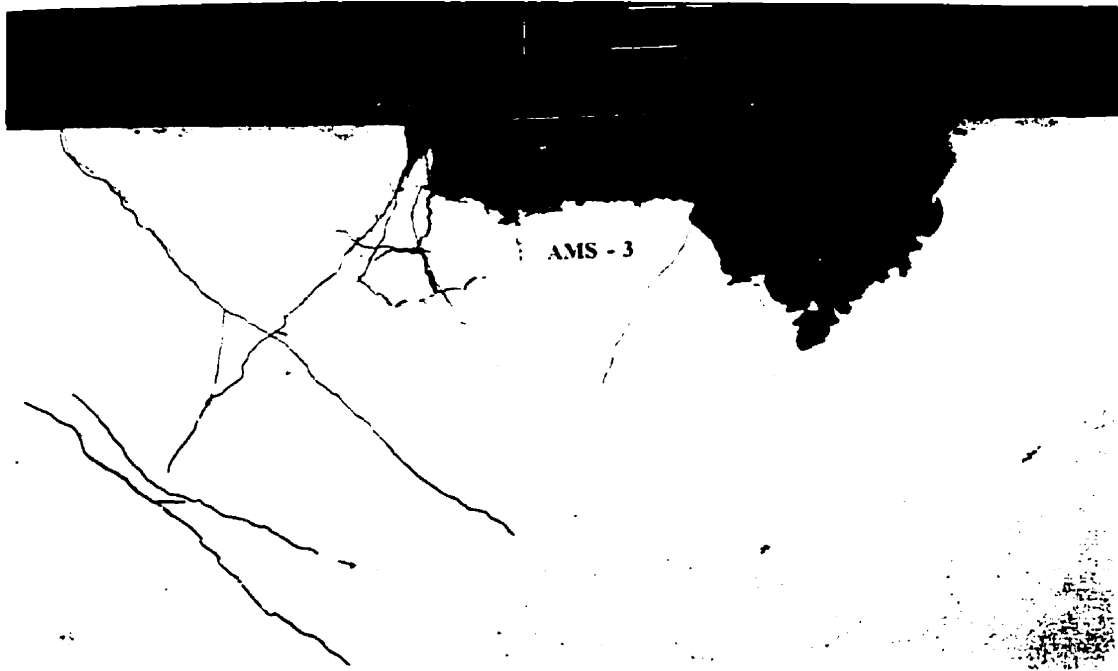
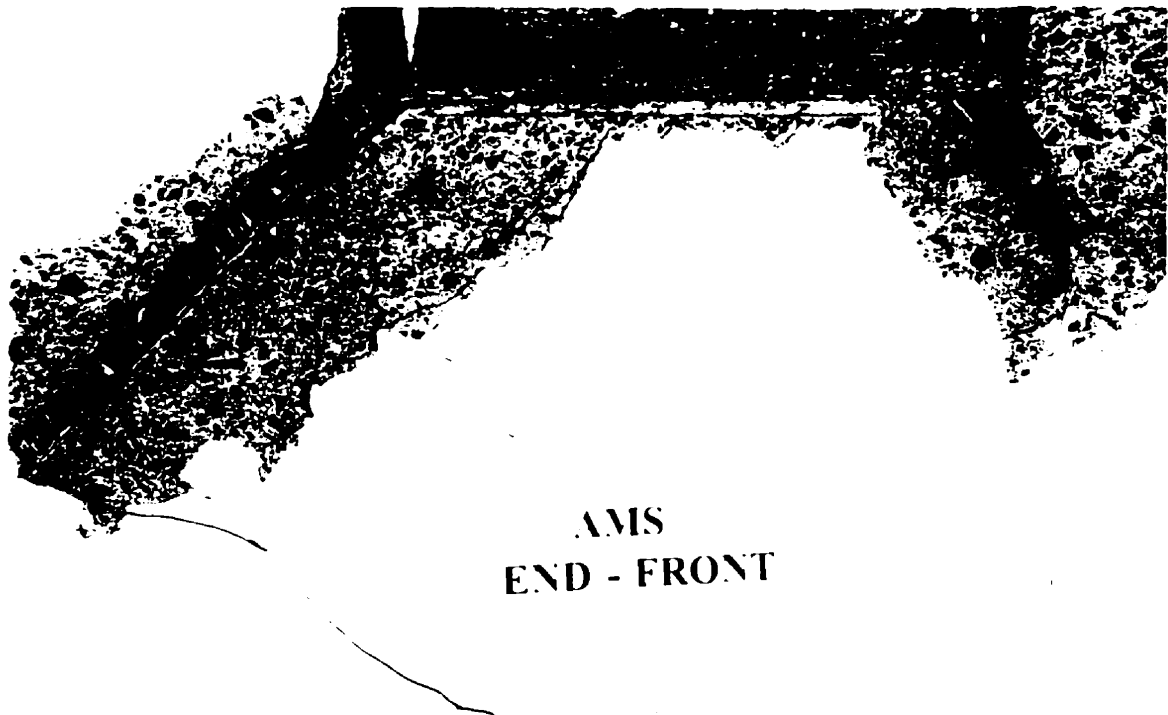


Figure 3.3 Strain distribution along the length of the bar prior to failure, Specimen AMS



(a) Connection at first yield, Δ_c , $P=110$ kN (Stage 3)



(b) Connection after testing

Figure 3.4 Photographs of Specimen AMS

3.2.2 Specimen AS

Specimen AS was the first specimen from Series A which was subjected to reversed cyclic shear loads only, as summarized in Table 3.1. The loading history and the resulting hysteresis response of the specimen are shown in Figure 3.5. The shear force versus the strain response for the bar anchor is shown in Figure 3.6. The photograph of the specimen at the yielding stage and after failure are shown in Figures 3.7(a) and 3.7(b), respectively. The following observations and behavioural aspects were noted during the testing and analyzing of the experimental results:

Cycles 1-3, 37 kN During first three elastic cycles, no cracks were noted. The load-deflection response during the earlier cycles was basically linear.

Cycles 4-6, 75 kN The first splitting crack appeared along the steel bar at a load of 62 kN, where nonlinear behaviour started.

Cycles 7-9, 105 kN The splitting crack along the bar extended with a measured crack width of 0.8 mm and a diagonal crack appeared perpendicular to the splitting crack. A symmetric crack pattern formed when the specimen was loaded in the opposite direction. During the following two cycles at the same load the diagonal crack and the splitting crack propagated, specially due to the reversed cyclic loads. The maximum measured crack width was 1.0 mm. During the last cycle of this load stage, the maximum strain at location 4 of the reinforcing bar was 2050×10^{-6} , which was close to the yield strain of the steel bar (see Figure 3.6(b)).

Cycles 10B-12A, 150 kN The anchor bar yielded during cycle 10B at a load of -150 kN, resulting in a sharp change in the stiffness as shown in Figure 3.5. The measured displacement at the first yield was $\Delta_y = -2.3$ mm. The photograph of the specimen at this stage is shown in Figure 3.8(a). During Cycle 11A at a load of 150 kN, the other steel leg yielded, which is confirmed by the strain of 2320×10^{-6} in Figure 3.7 (a). It should be mentioned that based on the hand method of the analysis (truss analogy), the reinforcing bar should have yielded at a load of 140 kN, but the yielding was delayed (it yielded at a load of 150 kN) since the surrounding concrete at front of the steel angle and along the reinforcing bar were resisting some of the tensile forces induced in the reinforcing legs. In the next cycle at the same load level, the strains at Locations 1 and

4 did not change considerably, while the displacements increased significantly. This caused the area under the hysteresis curve to increase in turn resulting in increase of the energy dissipated by the connection.

Cycles 12B, -140 kN This stage was implemented with a displacement control with $\Delta = 3\Delta_y = -7.4$ mm. The diagonal crack widened significantly (more than 8 mm), and the splitting crack widened to 7.4 mm and extended along the entire length of the bar, resulting in a drop of the load to -114 kN.

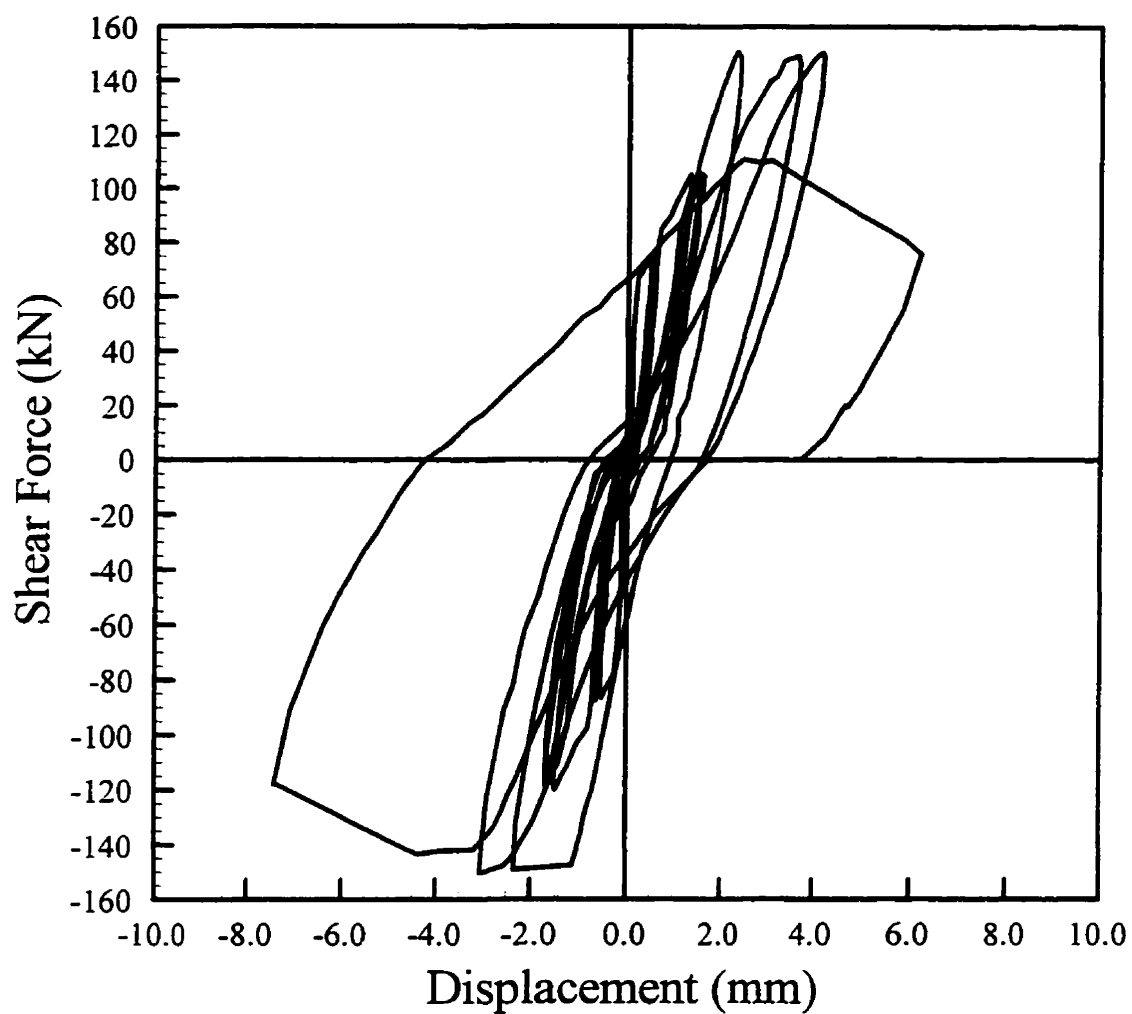
Cycles 13A, 110 kN Finally, the specimen was subjected to a maximum load of 110 kN (cycle 13A, $+2.1\Delta_y$), when the diagonal crack on the left leg widened significantly and a horizontal crack (connecting crack) formed to connect this crack to the previous diagonal crack on the right leg. The load then dropped to 75 kN with a measured displacement of 6.5 mm, resulting in the failure of the connection.

The profile of strains along the length of the reinforcing bar at the ultimate load (Figure 3.7) shows that more than two-third of the reinforcing steel bar had yielded. The 150 kN ultimate capacity was attained with the final mode of failure being due to diagonal cracking and splitting of the concrete along the bar length [Figure 3.8(b)]. The load-deflection response consisted of large loops specially in the last cycle, showing considerable dissipation of energy. The maximum measured displacement during testing was -7.4 mm (Cycle 12B), giving a displacement ductility ratio of $\mu = 7.4/2.3 = 3.2$.

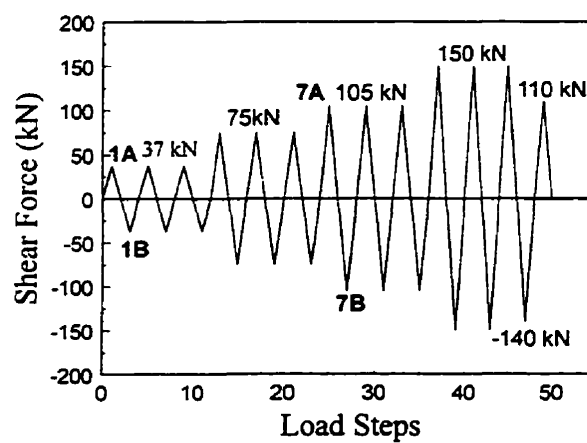
Table 3.1 Summary of Loading History for Specimen AS

Cycle	Load Stages	Horizontal Disp. (mm)*	Shear Force (kN)
Elastic	1A - 3B	+0.10 , -0.11	±38
Cracked	4A - 6B	+0.61 , -0.64	±77
Extensive cracking	7A - 9B	+1.61 , -1.64	±105
±1 Δ_y	10B - 11A	-2.3 , +3.0	±150
±1.5 Δ_y	11B - 12A	-3.4 , +4.2	±150
Failure, ±3 Δ_y	12B - 13A	-7.4 , +6.5	-140 , +110

* Maximum displacement at the peak load of the last cycle

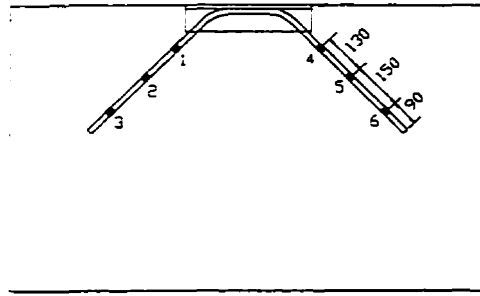


(a) Hysteresis response

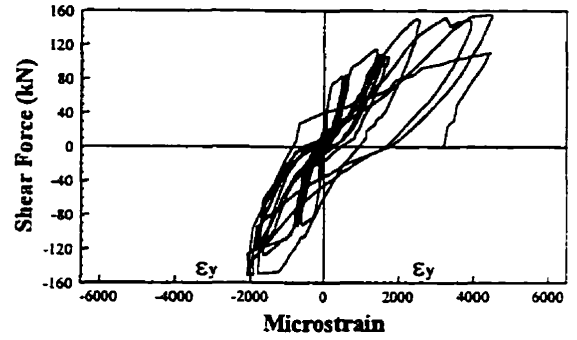


(b) Loading history

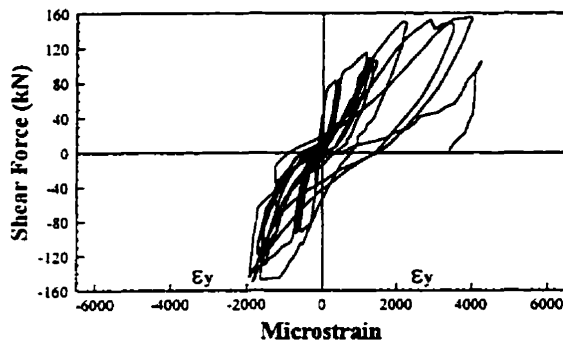
Figure 3.5 Hysteresis response and loading history for Specimen AS



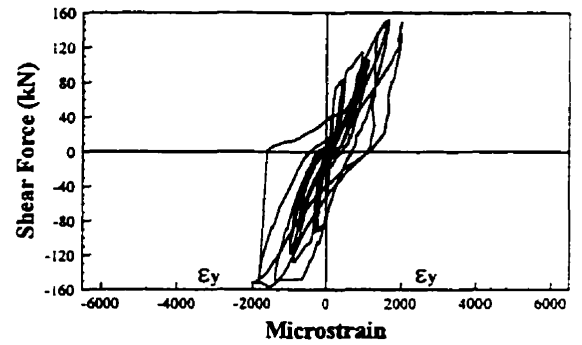
(a) Location of strain gauges



(b) Strains at Location 1



(c) Strains at Location 2



(d) Strains at Location 3

Figure 3.6 Measured strains along length of reinforcing anchor in Specimen AS

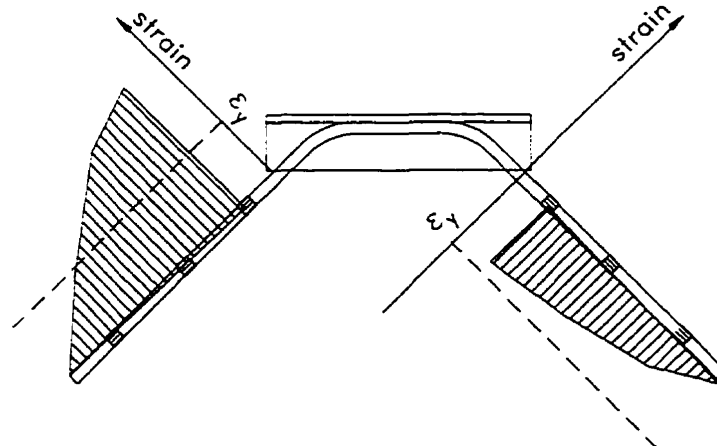
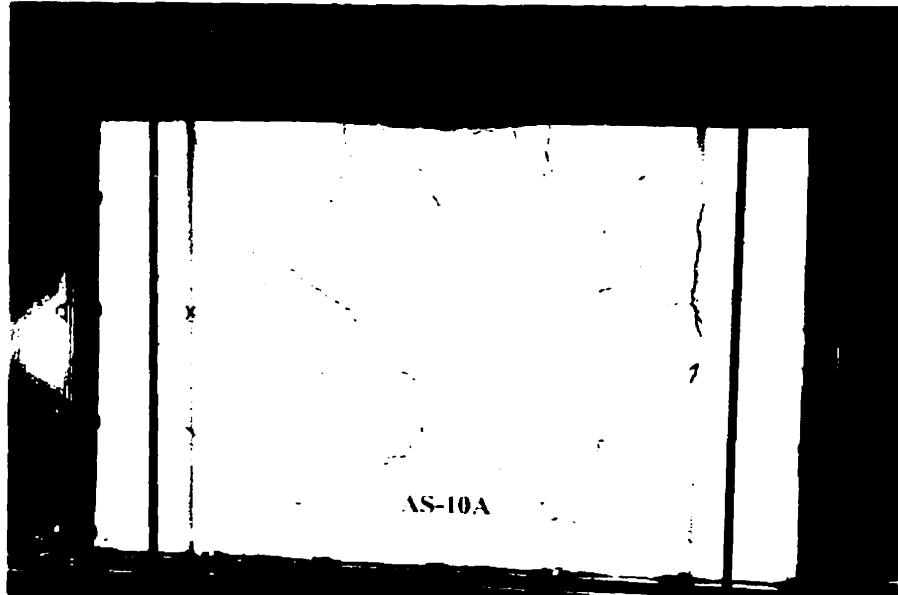
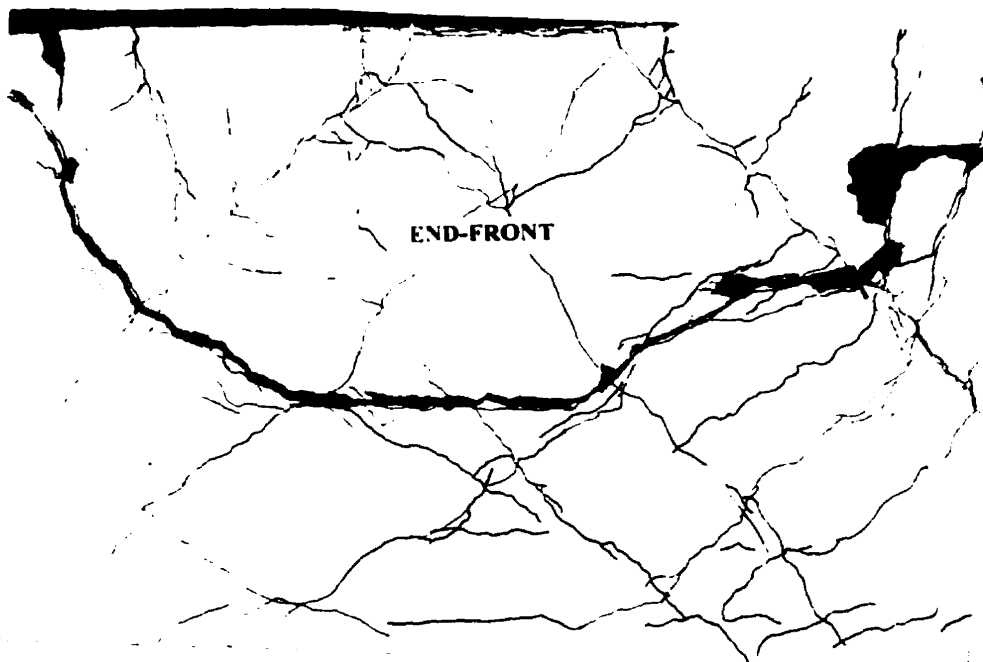


Figure 3.7 Strain distribution along the length of the bar prior to failure, Specimen AS



(a) Connection at first yield, Δ_s , $P=150$ kN



(b) Connection after testing

Figure 3.8 Photographs of Specimen AS

3.2.3 Specimen AT

The Specimen AT was tested under the reversed axial loading only, as shown in Table 3.2. Figure 3.9 shows the hysteresis response and the loading history for Specimen AT. The applied axial load versus the strain response at different locations on the steel bar anchor are shown in Figure 3.10. The distribution of the strains along the length of the reinforcing bar prior to failure is shown in Figure 3.11. The photograph of the connection at the yield stage and after failure are presented in Figures 3.12(a) and 3.12(b), respectively. The following behavioural aspects were noted based on the observations made during the testing and analysis of the experimental results:

Cycles 1-3, 30 kN The response of the connection during the first three cycles ($P=\pm 30$ kN) was basically linear elastic.

Cycles 4-6, 65 kN As the load was increased to 65 kN (Cycle 4A), a splitting crack appeared and extended along the reinforcing bar legs ($w_{\max}=0.15$ mm), while a horizontal crack ($w_{\max}=0.08$ mm) formed almost at the steel bar end. This caused a reduction in the stiffness of the connection and a significant increase in the displacement, resulting in inelastic response (see Figure 3.9, Cycle 4A). The displacements were not completely recovered during the compression loading cycles, which demonstrated stiffer response accompanied with the closing of the cracks. During two other cycles at the same load, the displacement increased during tensile loading, especially in the last cycle, and it was recovered when the compression loads were applied. The diagonal and splitting cracks propagated more due to the cyclic loading. The maximum measured displacement and crack width were 3 mm and 0.33 mm, respectively.

Cycles 7-9, 95 kN As the applied load was increased to 95 kN (cycle 7A), the splitting crack along the bar legs widened more and extended further, and more diagonal cracks appeared perpendicular to the splitting crack. As can be seen from Figure 3.2, the displacement increased significantly by more than twice the displacement at the end of stage 6A, indicating energy dissipation in the connection. The measured strain at Location 1 was 2150×10^{-6} which was about the yield strain of the steel anchor, see Figure 3.10(b). In the following cycle of this load stage (cycle 8A), the reinforcing bar yielded (Figure 3.10b), while the measured displacement was $\Delta_y = 4.2$ mm. Figure 3.12(a) shows

Table 3.2 Summary of Loading History for Specimen AT

Cycle	Load Stages	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.4 , +0.1	±30
Cracked	4A - 6B	+3.0 , +1.6	±67
Extensive Crack	7A - 7B	+4.1 , +1.4	±95
±1Δ _y	8A - 9B	+4.2 , +1.3	±95
Failure, 1.4Δ _y	10A	+5.6	+106

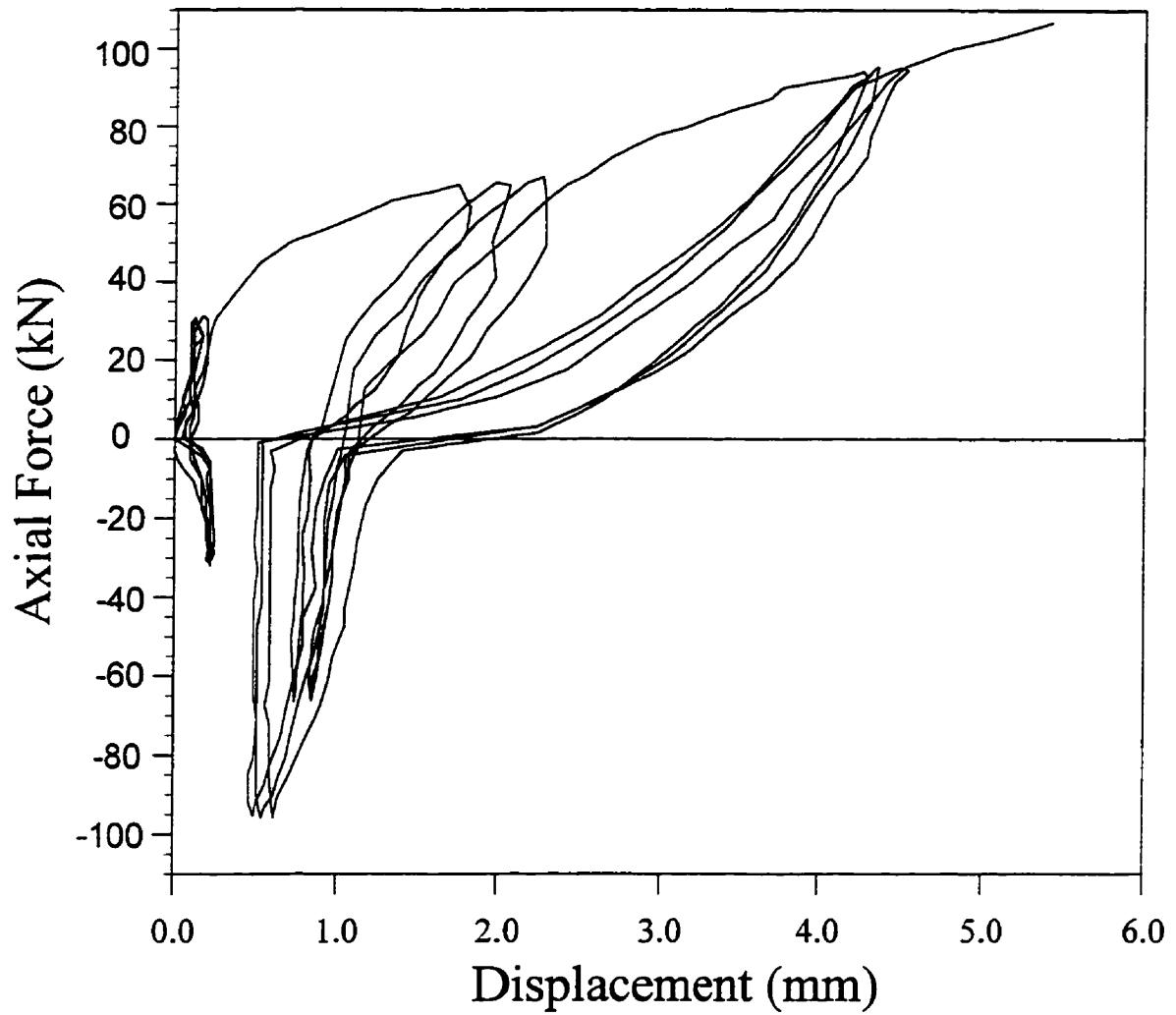
* Maximum displacement at the peak load of the last cycle

a photograph of the specimen at the yield stage.

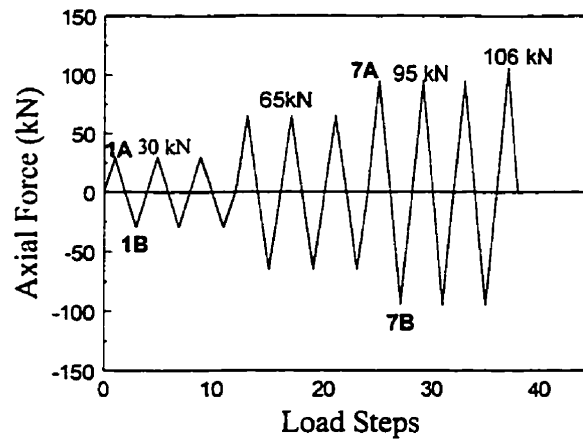
Cycles 10, 106 kN Finally, the specimen was subjected to a load of 106 kN, when the ultimate capacity was attained (cycle 10A). The final mode of the failure was due to splitting and spalling of the concrete along the bar length [Figure 3.12(b)]. The maximum axial displacement of the connection was measured to be 5.6 mm, giving a displacement ductility ratio of 1.4 .

Comparing the response under the tensile and compressive axial loads, the load deflection response consisted of large loops under tensile loads and small loops in compression loading, which is due to the deterioration of bond (splitting crack) and formation of diagonal cracks during tensile loading and closing of the cracks during the compressive loading, respectively. The energy dissipated and the residual displacement are small due to the small area under the loops in the compression portion of load-deflection curve.

The variation of the strains along the length of the reinforcing bar at the ultimate load (Figure 3.11) indicates that approximately one third of the steel bar length had yielded when the connection was subjected to tensile loads.

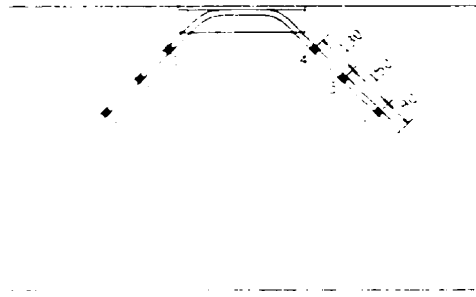


(a) Hysteresis response

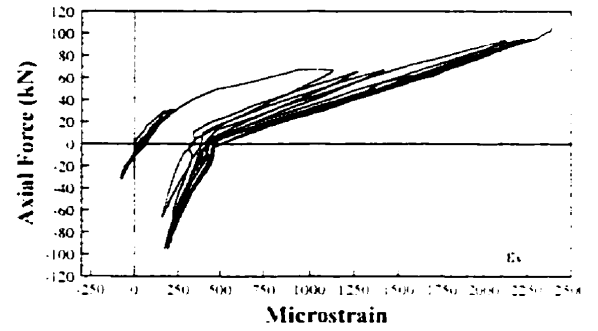


(b) Loading history

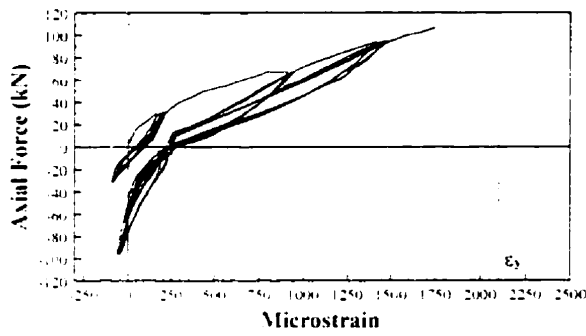
Figure 3.9 Hysteresis response and loading history for Specimen AT



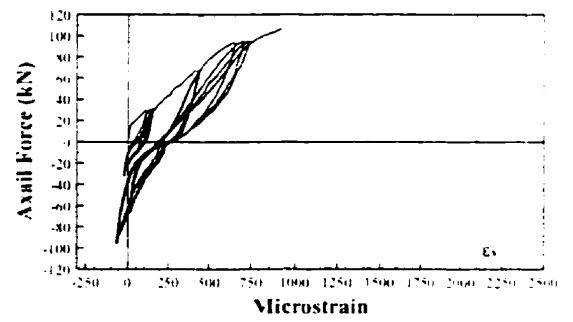
(a) Location of strain gauges



(b) Strains at Location 1



(c) Strains at Location 2



(d) Strains at Location 3

Figure 3.10 Measured strains along length of reinforcing anchor in Specimen AT

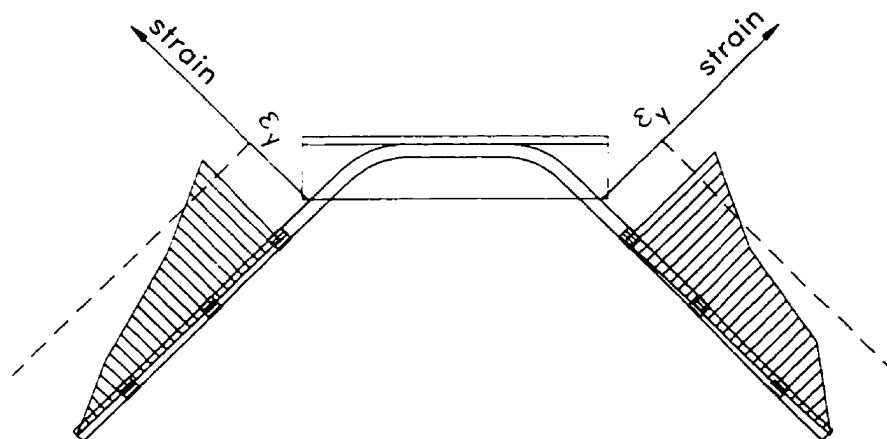
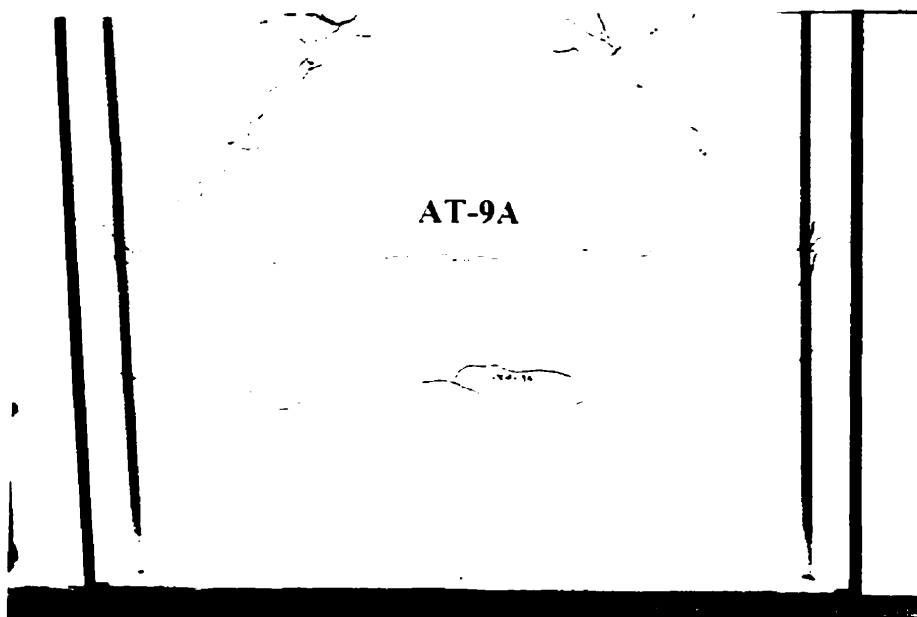
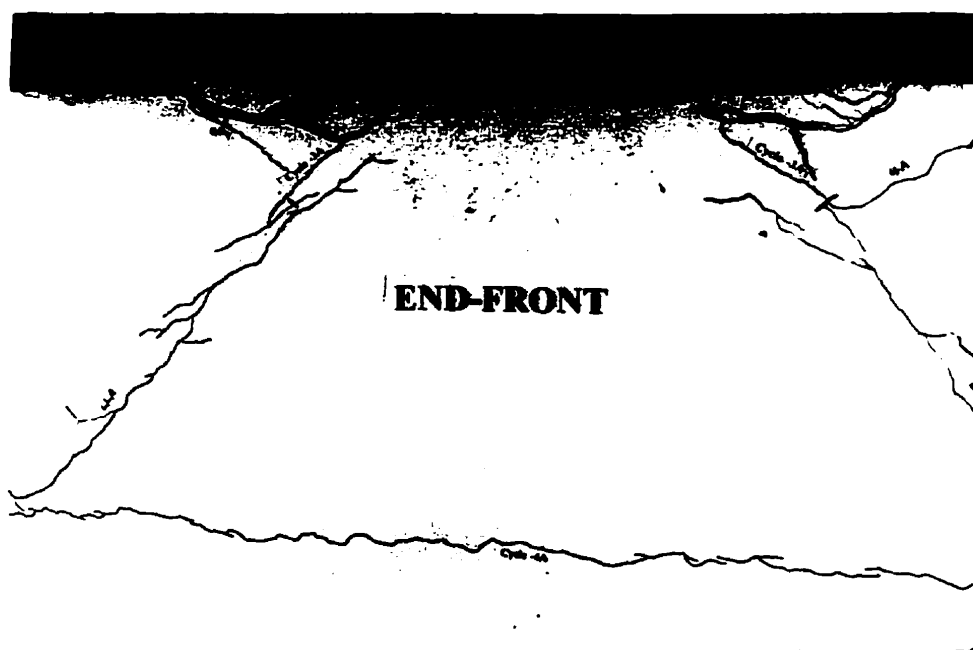


Figure 3.11 Strain distribution along the length of the bar prior to failure, Specimen AT



(a) Connection at first yield, Δ_y , $P=95$ kN



(b) Connection after testing

Figure 3.12 Photographs of Specimen AT

3.2.4 Specimen AST1

In order to investigate the response of the connections to combined axial and shear forces, Specimen AST1 was subjected to combined cyclic shear and axial loads. The ratio of axial to shear loads (T/V) was 0.7. The loading histories and the load-deflection responses for the combined cyclic shear and axial loadings are shown in Figures 3.13 and 3.14, respectively. The measured strains along the length of the reinforcing bar versus the shear and axial loads are presented in Figures 3.15 and 3.16. Figures 3.17(a) and 3.17(b) show the photographs of the specimens at the yielding stage and after failure, respectively. The distribution of strains with distance along the length of the anchor bar at the ultimate load is shown in Figure 3.18. The following behavioural aspects were noted during the testing and from the analysis of the experimental results.

Cycles 1-3, $V=20\text{ kN}$ $T=40\text{ kN}$ The slight deflection at zero axial load were caused by initial application of shear load in imperfect closing of cracks. The linear responses for both axial and shear loadings were observed in the first steps of loading (Figures 3.13 and 3.14).

Cycles 4-6, $V=45\text{ kN}$ $T=32\text{ kN}$ As the load was increased to 45 kN and 32 kN for shear and axial forces, respectively, the splitting cracks appeared along the reinforcing bar, when the nonlinear responses started (Figures 3.13 and 3.14). Up to the end of this load stage, the shape of shear and axial responses were alike, and the area under the load-deflection responses were not notable.

Cycles 7-9, $V=65\text{ kN}$ $T=45\text{ kN}$ Following the load Cycle 7, the splitting cracks widened and extended along the anchor legs, and a significant horizontal crack formed and extended to the compression zone. These cracks caused a significant decrease in the connection stiffness resulting in an increase in the displacements and the area under the load-deflection responses (Figures 3.13 and 3.14).

Cycles 10, $V=88\text{ kN}$ $T=62\text{ kN}$ The reinforcing bar yielded during Cycle 10A under the combined shear and tensile loading (Figures 3.15b and 3.16b). The measured displacements for both shear and axial loads at yielding was 1 mm (Δ_y). The photograph in Figure 3.17 (a) shows the test specimen at yielding. Once yielding developed, significant deterioration of the concrete around the tensile leg was observed. In this case,

Table 3.3 Summary of Loading History for Specimen AST1

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.11,-0.03	±20	+0.04,-0.08	±15
Cracked	4A - 6B	+0.32,-0.22	±45	+0.48,-0.08	±32
Extensive cracking	7A - 9B	+0.82,-0.45	±65	+0.81,-0.42	±45
±1 Δ_y	10A - 10B	+1.06,-0.62	±88	+1.07,-0.84	±62
±2 Δ_y , Failure	11A	+2.02	+88	+2.61	+40

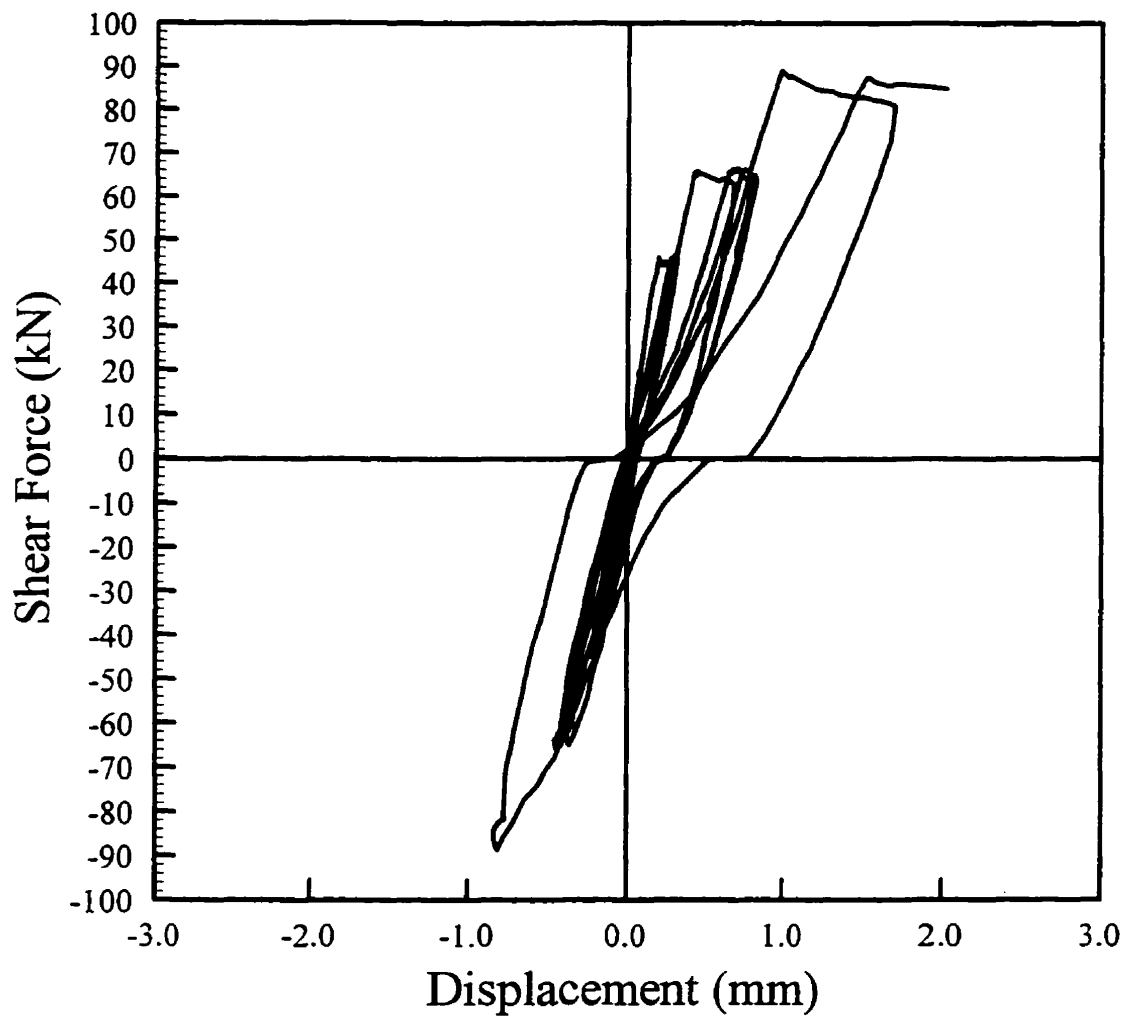
* Maximum displacement at the peak load of the last cycle

it was evident that the tensile leg was controlling the basic failure mode [Figure 3.17]. Cycles 11, $V=88$ kN $T=40$ kN This stage was implemented using displacement control with measured displacement equal to $2\Delta_y$. The specimen failed during application of combined load of 88 kN and 40 kN for shear and axial forces, respectively. The axial response was basically stiffer during the compression loadings, however, due to the shear force, the response of the connection in both directions were softer compared with that of the Specimen AT. Small loops (pinching) and stiff response were observed in the compression loops due to closing of the cracks (Figure 3.14).

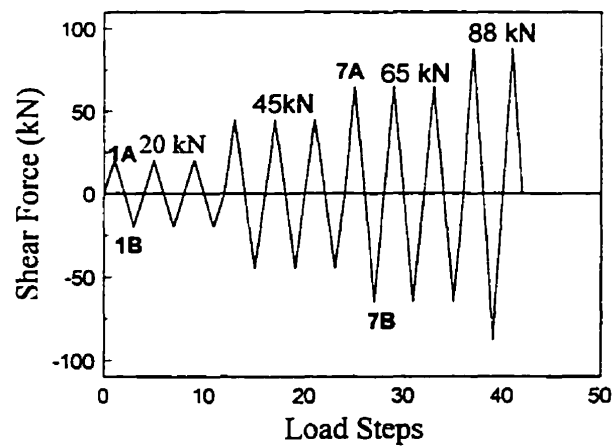
The ultimate shear and axial strengths of the connection subjected to combined forces were 88 kN and 62 kN, respectively. The ultimate horizontal (shear) and axial displacements were measured to be 2 mm and 2.6 mm, respectively, giving a displacement ductility ratio of 2. Failure occurred due to the complete deterioration of concrete close to the free edge and with the complete splitting of the concrete (splitting bond failure) along the leg in the tension zone subjected to high tensile stresses resulting in slipping of the steel bar in the tension zone [Figure 3.17(b)]. This mode of failure was close to that observed in Specimen AT.

The variation of the strains in the steel bar along its length at the ultimate load, indicates that more than half of the reinforcing bar had yielded (Figure 3.18).

The results show that under combined cyclic axial and shear loads ($T/V=0.7$), the strength of the connection reduced to about 59% and 58% of those for pure shear and for pure axial forces, respectively. This drop in the shear resistance is due to the weakening of the bond strength due to cyclic axial load for the succeeding application of the shear forces. In general, the shear hysteresis response was different from that for Specimen AS due to the effect of the axial forces. This caused larger loops in the positive direction (combined shear and tensile loads). The same load-deflection pattern was noted for axial response compared with the response of the Specimen AT. The displacements had increased compared with the response for the Specimens AS (pure shear) and AT (pure axial force) at a similar load stage. In fact, this increase of the displacement was caused by the application of the axial load. The crack patterns were different from those for the Specimens AS and AT.

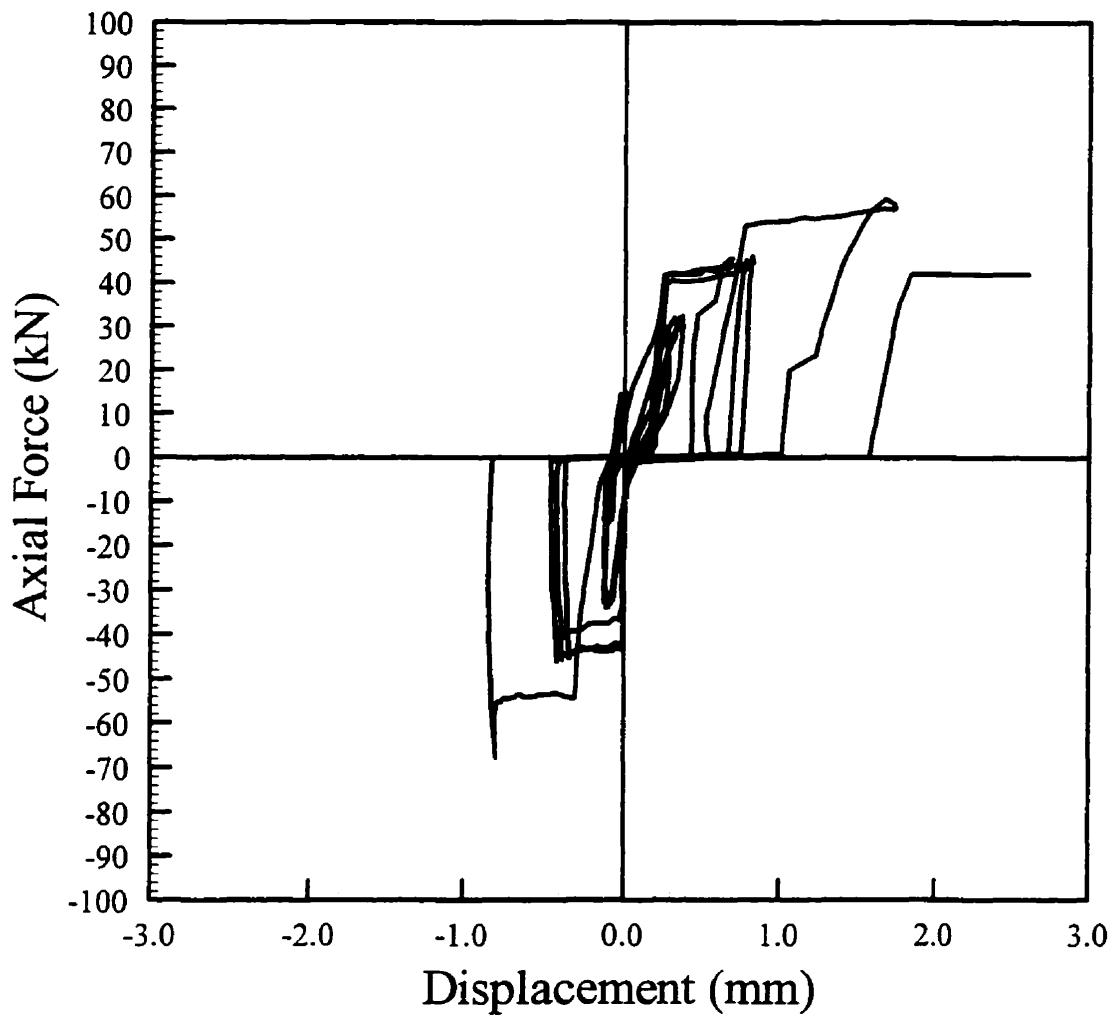


(a) Hysteresis response

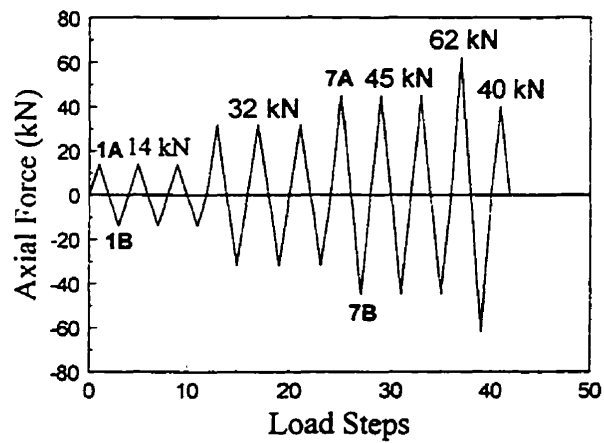


(b) Loading history

Figure 3.13 Hysteresis shear response and loading history for Specimen AST1

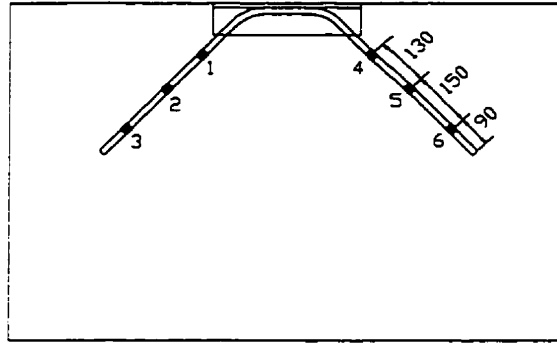


(a) Hysteresis response

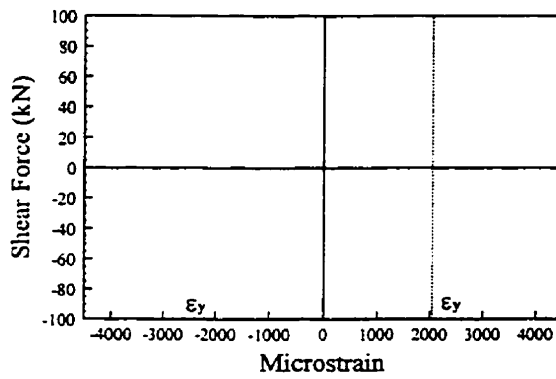


(b) Loading history

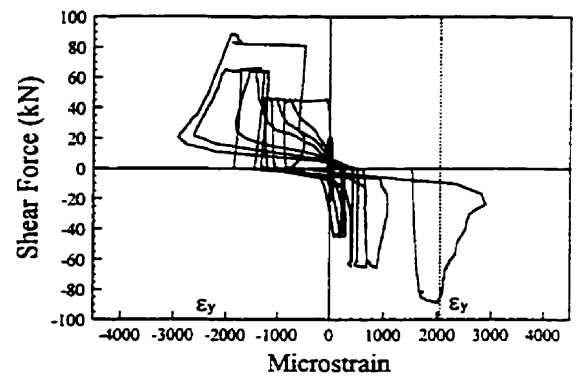
Figure 3.14 Hysteresis axial response and loading history for Specimen AST1



(a) Location of strain gauges

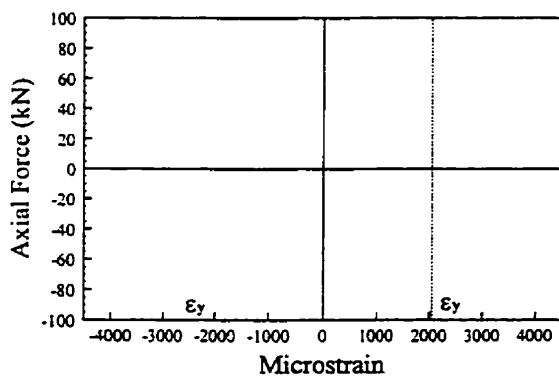


(b) Strains at Location 1, Shear response

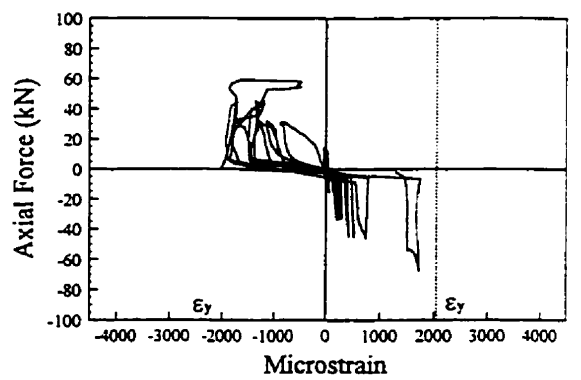


(c) Strains at Location 4, Shear response

Figure 3.15 Strains along length of anchor in Specimen AST1(Shear response)

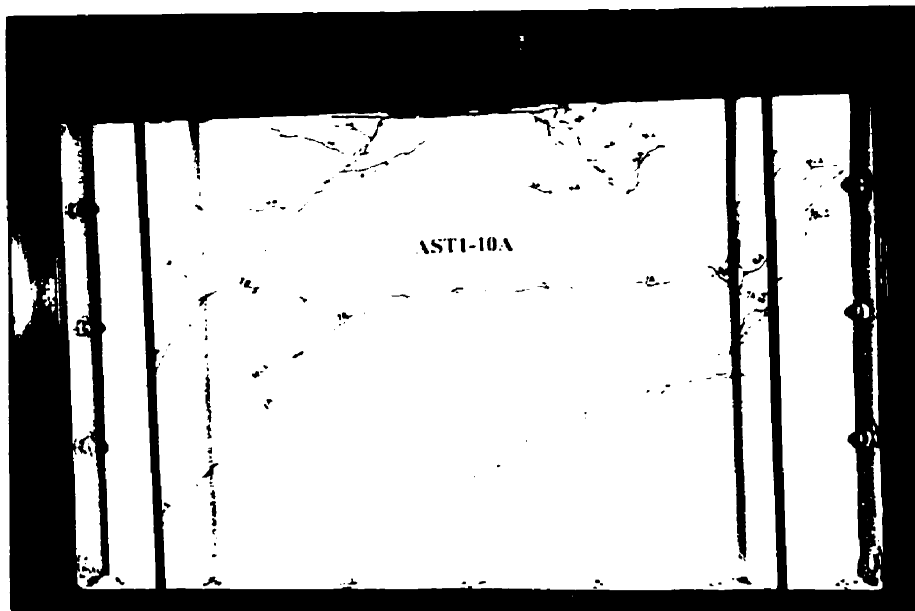


(b) Strains at Location 1, Axial response

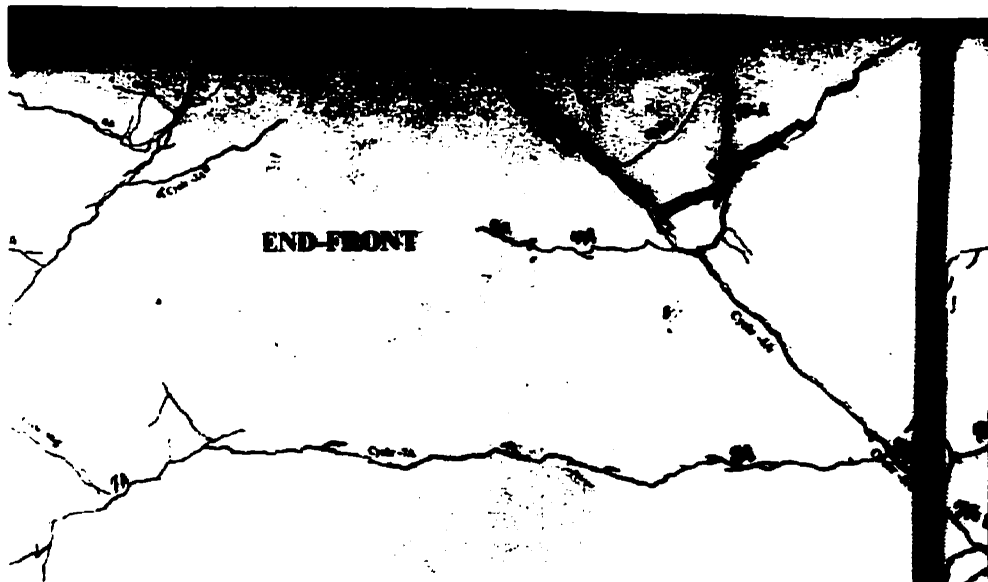


(c) Strains at Location 4, Axial response

Figure 3.16 Strains along length of anchor in Specimen AST1(Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}}=88 \text{ kN}$, $P_{\text{axial}}=62 \text{ kN}$



(b) Connection after testing

Figure 3.17 Photographs of Specimen AST1

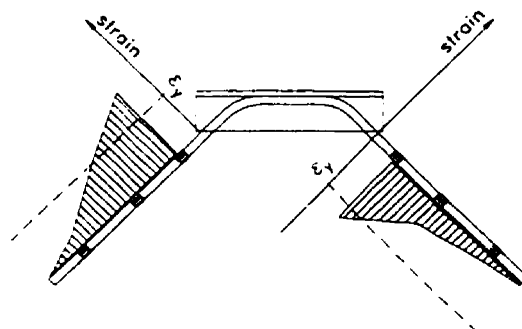


Figure 3.18 Strain distribution along the length of bar prior to failure, Specimen AST1

3.2.5 Specimen AST2

Specimen AST2 was loaded under combined cyclic shear and axial forces with T/V ratio = 1.0. The loading histories and the load-deflection responses for cyclic shear and axial loading are presented in Figures 3.19 and 3.20, respectively. The strains at different locations along the length of the anchor bar for shear and axial loads are shown in Figures 3.21 and 3.22, respectively. The photographs of the connection at the yielding stage and after failure are presented in Figures 3.23(a) and 3.23(b), respectively. The profile of the strains with distance along the length of anchor bar at the ultimate load is shown in Figure 3.24

Cycles 1-3, $V=T=21$ kN At early stages of load cycles, the response for both shear and axial were basically linear elastic (Figures 3.19 and 3.20).

Cycles 4-6, $V=T=46$ kN The first cracks appeared along the tensile leg, when the specimen was subjected to a tensile load of 46 kN and an axial load of 30 kN; which was increased to 46 kN for this load stage (Cycle 4). Upon cracking, the slope of the axial load-deflection decreased, and inelastic response started. In the subsequent cycles at this load level, one major diagonal crack and several minor cracks perpendicular to the splitting crack were observed, and the splitting crack along the reinforcing bar extended toward the reinforcing bar end.

Cycles 7-9, $V=T=60$ kN As the load was increased to 60 kN for both shear and axial forces (Cycle 7A), the density of cracks increased all over the specimen. The loss of tensile strength in the concrete along the steel bar (splitting bond crack) is indicated by a notable increase in the displacement prior to yielding. During Cycle 9A, spalling of the concrete in the tensile zone indicated degradation of the concrete at that location. The maximum measured horizontal and axial displacements were 1.3 mm and 1.5 mm, respectively. During this cycle, the splitting and diagonal cracks widened; the anchor steel bar in the tensile zone was also visible close to the free edge.

Cycle 10, $V=73$ kN $T=73$ kN The reinforcing bar at Location 1 yielded during Cycle 10A, when the shear load was 73 kN and the axial load was 69 kN. Following the increase of the axial load to 75 kN, the reinforcing bar yielded at Location 2 along the steel bar (Figures 3.21 and 3.22). The measured displacements at the yielding of

Table 3.4 Summary of Loading History for Specimen AST2

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.14,-0.08	±21	+0.06,-0.05	±21
Cracked	4A - 6B	+0.7,-0.2	±46	+0.9,-0.2	±46
Extensive cracking	7A - 9B	+1.3,-0.4	±60	+1.5,-0.2	±60
±1Δ _y	10A	+1.8	+73	+1.6	±75
+1.6Δ _y , Failure	10A	+2.9	+75	+2.0	+75

* Maximum displacement at the peak load of the last cycle

steel were 1.8 mm and 1.7 mm for horizontal and axial loads, respectively. Figure 3.24(a) shows photograph of the specimen at yielding, indicating significant cracking of the specimen, especially in tensile zone.

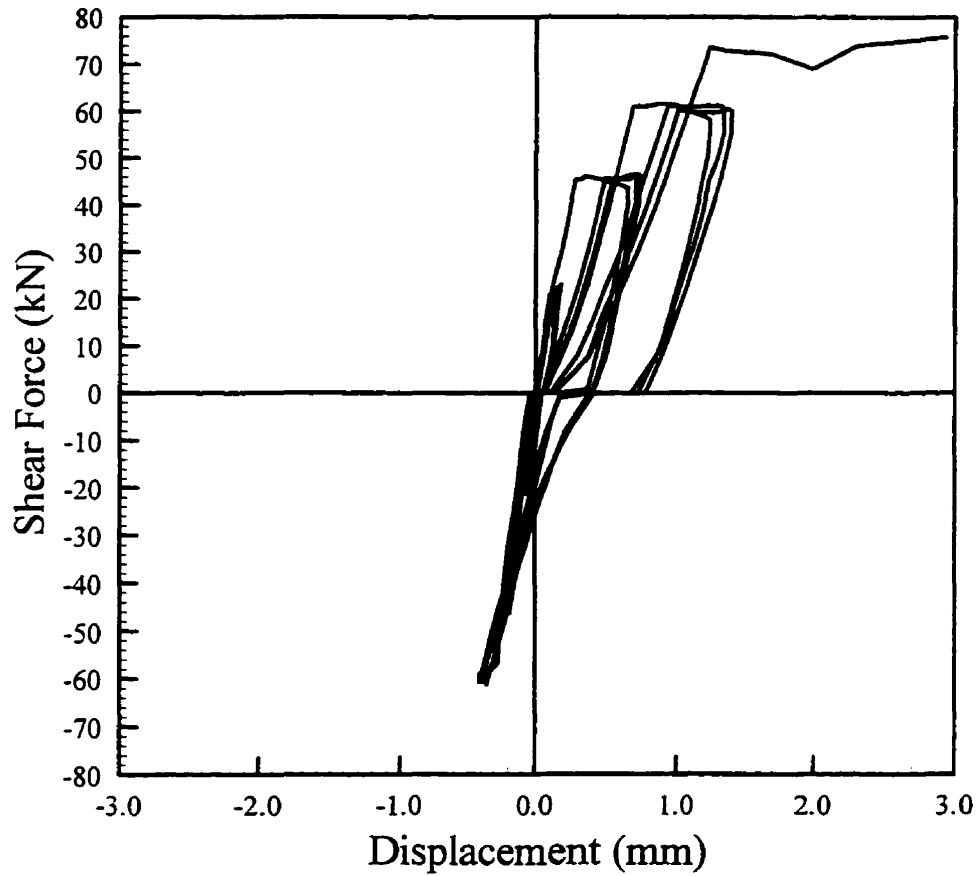
Finally, the maximum strength of 75 kN for both of shear and axial loading were achieved. The ultimate horizontal and axial displacements were measured to be 2.9 mm and 2.0 mm, respectively, giving a maximum displacement ductility ratio of 1.6.

Due to complete degradation of the concrete, the connection lost most of its strength. The test was stopped at the end of this stage as soon as the load dropped below 75 percent of the ultimate capacity to avoid damage to the instrumentation. The mode of failure was due to splitting of the concrete along the tensile leg (splitting bond failure), stressed by combined shear and tensile axial force. Photograph of the specimen after testing explains this failure mode, see Figure 3.23(b).

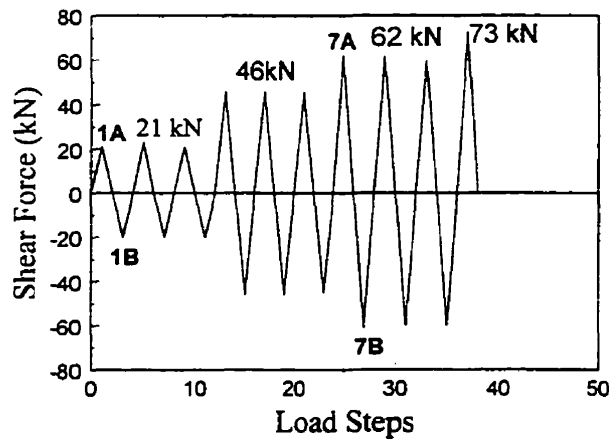
The profile of the measured strains along the reinforcing bar shows that more than two-third of the reinforcing bar in the tensile zone had yielded (Figure 3.24). No yielding was noted in the compression area, i.e. at Locations 4, 5, or 6.

Generally, a small shift of displacement at zero load in the axial load-deflection response was noted. Most of this shift is attributed to the shear load application, the opening of the crack and the subsequent imperfect closing. A comparison of the shear

hysteresis response with that for Specimen AS, shows that it is different due to the effect of the axial loads. The large loops in the positive loading direction (combined shear and tensile loads) and very narrow in negative direction due to compressive axial load were observed. The same load-deflection pattern was noted for the shear response compared with the response of the Specimen AST1; the most important difference was related to the pattern in the negative direction which for Specimen AST2 is very narrow due to the higher compressive axial forces. The axial load-deflection pattern was similar to that for the shear response, which is different from those observed for the Specimens AST1 and AT. Again, this implies that the effect of the shear forces on the axial behaviour causes a different pattern response. The displacement in the positive direction had increased compared with those for Specimen AS (pure cyclic shear load), AT (pure cyclic axial load) and AST1 ($T/V=0.7$). The cracking pattern was similar to that for the Specimen AST1 and different from those for the Specimens AS and AT. The area under the the loops of load-deflection curves for both axial and shear loads were not very large, which indicates that the amount of energy dissipated through the cycles was not significant.

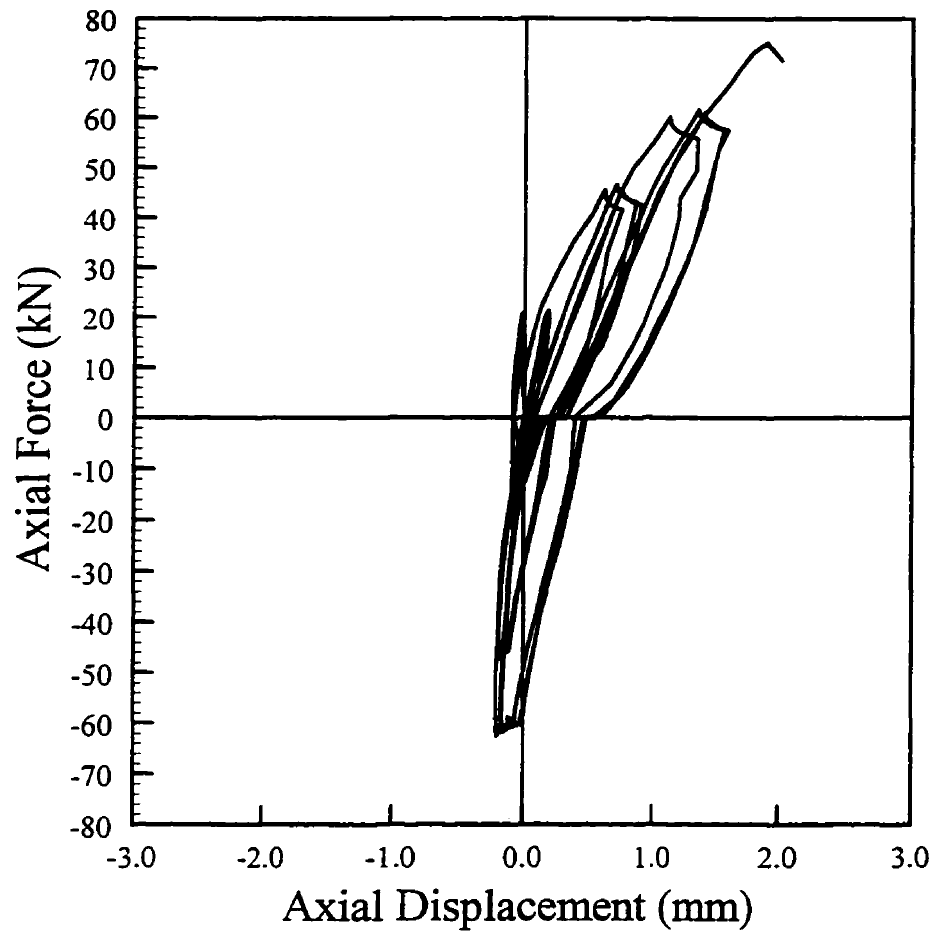


(a) Hysteresis response

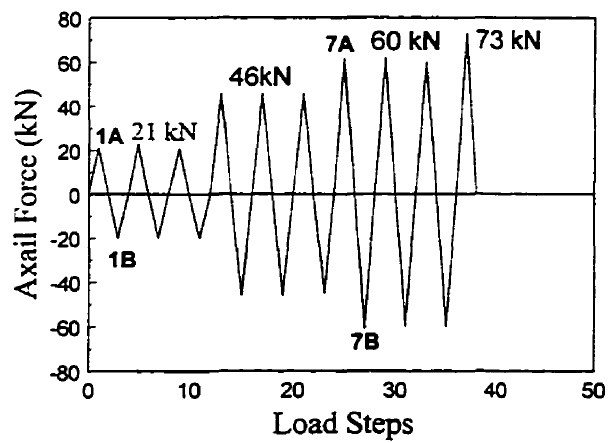


(b) Loading history

Figure 3.19 Hysteresis shear response and loading history for Specimen AST2

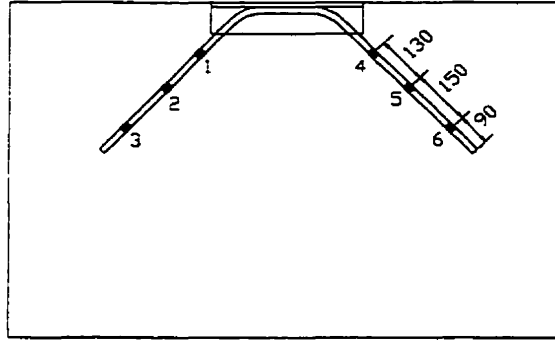


(a) Hysteresis response

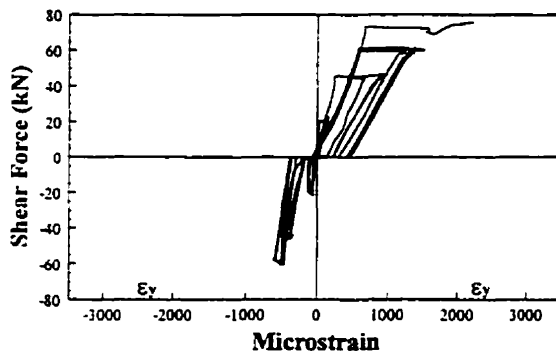


(b) Loading history

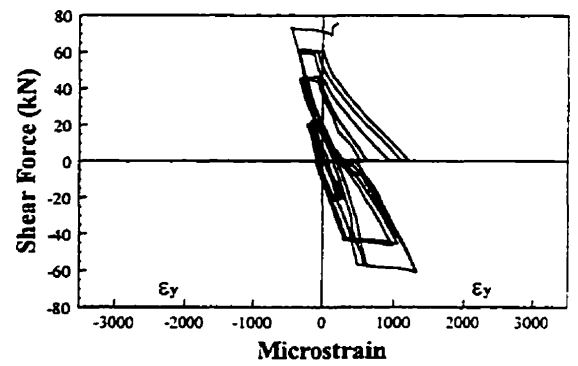
Figure 3.20 Hysteresis axial response and loading history for Specimen AST2



(a) Location of strain gauges

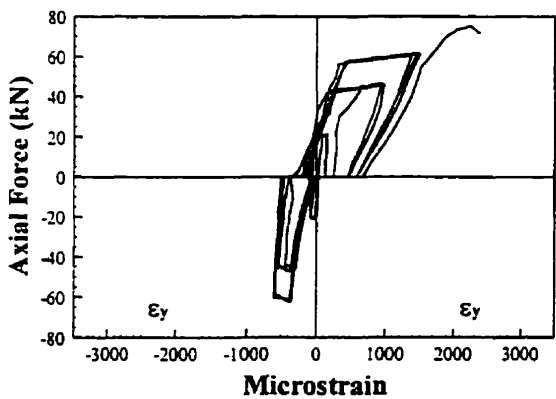


(b) Strains at Location 1, Shear response

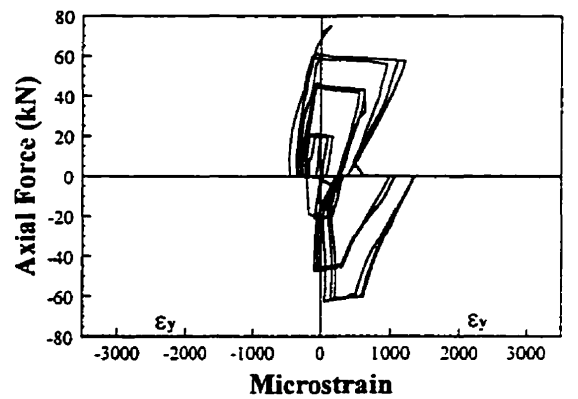


(c) Strains at Location 4, Shear response

Figure 3.21 Strains along length of anchor in Specimen AST2 (Shear response)

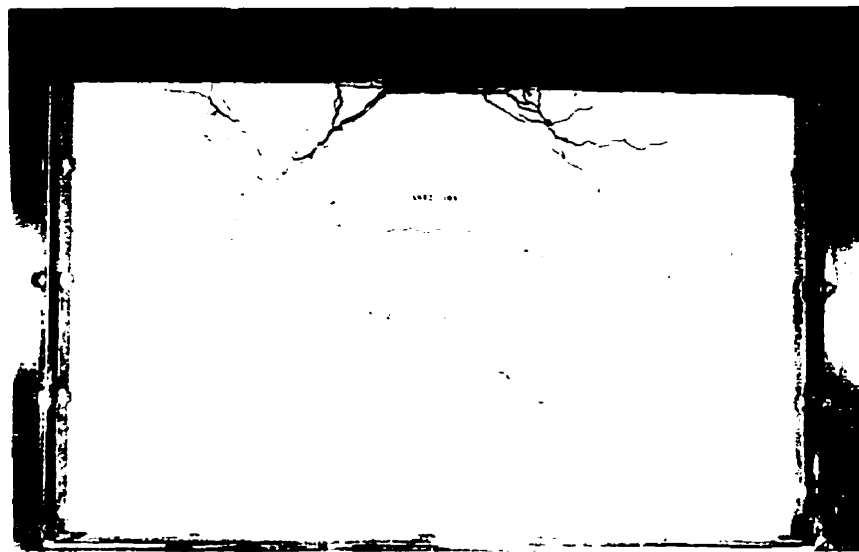


(b) Strains at Location 1, Axial response

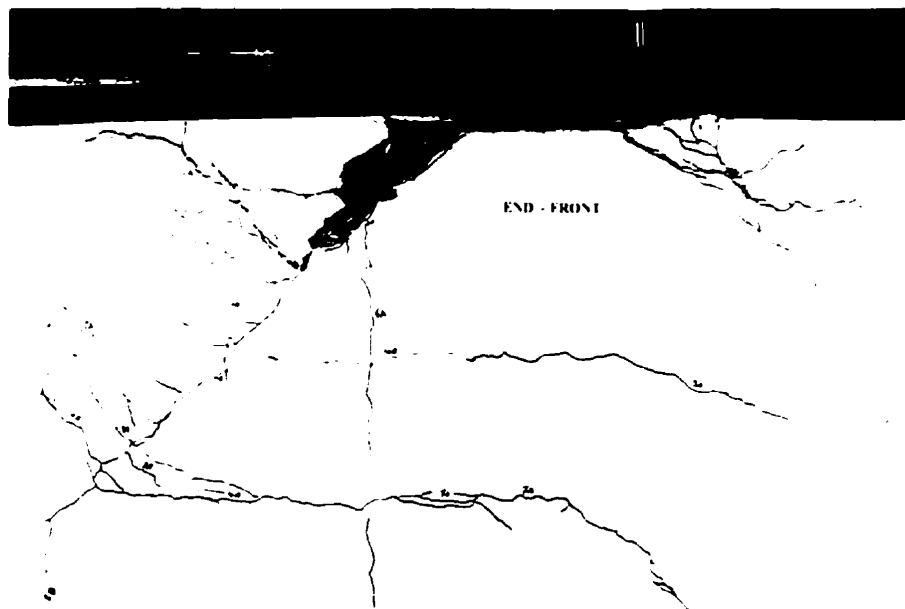


(c) Strains at Location 4, Axial response

Figure 3.22 Strains along length of anchor in Specimen AST2 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}} = P_{\text{axial}} = 73 \text{ kN}$



(b) Connection after testing

Figure 3.23 Photographs of Specimen AST2

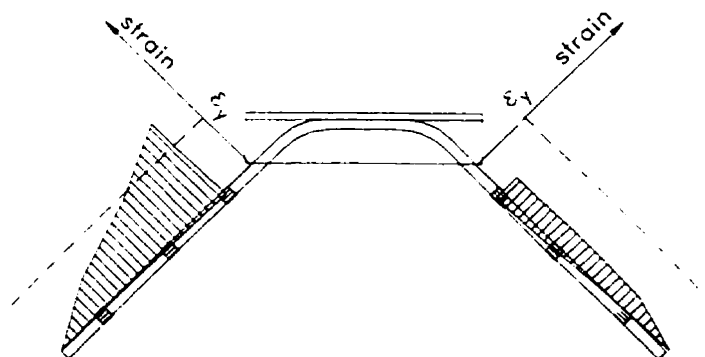


Figure 3.24 Strain distribution along the length of bar prior to failure, Specimen AST2

3.2.6 Specimen AST3

The purpose of testing Specimen AST3 was to investigate the connection behaviour under combined seismic simulated shear and the low axial forces; the ratio of axial force to shear forces was 0.3. Table 4.5 summarizes the loading histories for the specimen. Figures 3.25 and 3.26 show the loading histories and the hysteresis curve for cyclic shear and axial loading, respectively. The measured strains in the steel anchor bar for the shear and axial forces are presented in Figures 3.27 and 3.28, respectively. Figures 3.29(a) and 3.29(b) show the photographs of the test specimen at the yielding stage and after failure, respectively. The distribution of the strains along the length of steel bar at the ultimate load is shown in Figure 3.30. The following behavioural aspects were noted during testing and the analysis of the experimental results:

Cycles 1-3, $V=28$ $T=9$ kN The specimen was loaded through three elastic cycles (load Cycles 1-3) at a peak load of 28 kN and 9 kN for shear and axial loads, respectively. The response for both shear and axial were noted to be linear elastic (Figures 3.25 and 3.26). No cracks were visible.

Cycles 4-6, $V=60$ $T=20$ kN The first cracks observed along the tensile leg, when the specimen was subjected to a tensile load of 60 kN and an axial load of 20 kN (Cycle 4). Upon cracking, the slope of load-deflection decreased resulting in an inelastic response (Figures 3.25 and 3.26). In subsequent cycles at this load stage, several minor cracks appeared perpendicular to the splitting crack in the tensile zone and a splitting crack formed along the other leg of the bar. The maximum measured crack width was 0.3 mm.

Cycles 6-9, $V=85$ $T=27$ kN As the load was increased to 85 kN and 27 kN for shear and axial forces, respectively (Cycle 7A), many new cracks appeared which propagated all over the specimen. This caused a notable increase in the displacement prior to the yielding of the reinforcing bar. During the last cycle of this load stage (Cycle 9), the widening of the splitting and diagonal cracks increased; the maximum crack width was measured to be 0.6 mm. The maximum measured horizontal and axial displacement were 1.1 mm and 0.67 mm, respectively. Up to this stage, the area under the loops was not large resulting in small energy dissipation through the load cycles.

Table 3.5 Summary of Loading History for Specimen AST3

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.14,-0.16	±28	+0.06,+0.12	±9
Cracked	4A - 6B	+0.5,-0.8	±60	+0.6,+0.4	±20
Extensive cracking	7A - 9B	+1.1,-1.2	±85	+0.7,+0.5	±27
±1 Δ_y	10A - 12B	+2.2,-2.2	±115	+0.9,+0.9	±36
±2 Δ_y , Failure	13A	+3.3	+140	+1.25	+29

* Maximum displacement at the peak load of the last cycle

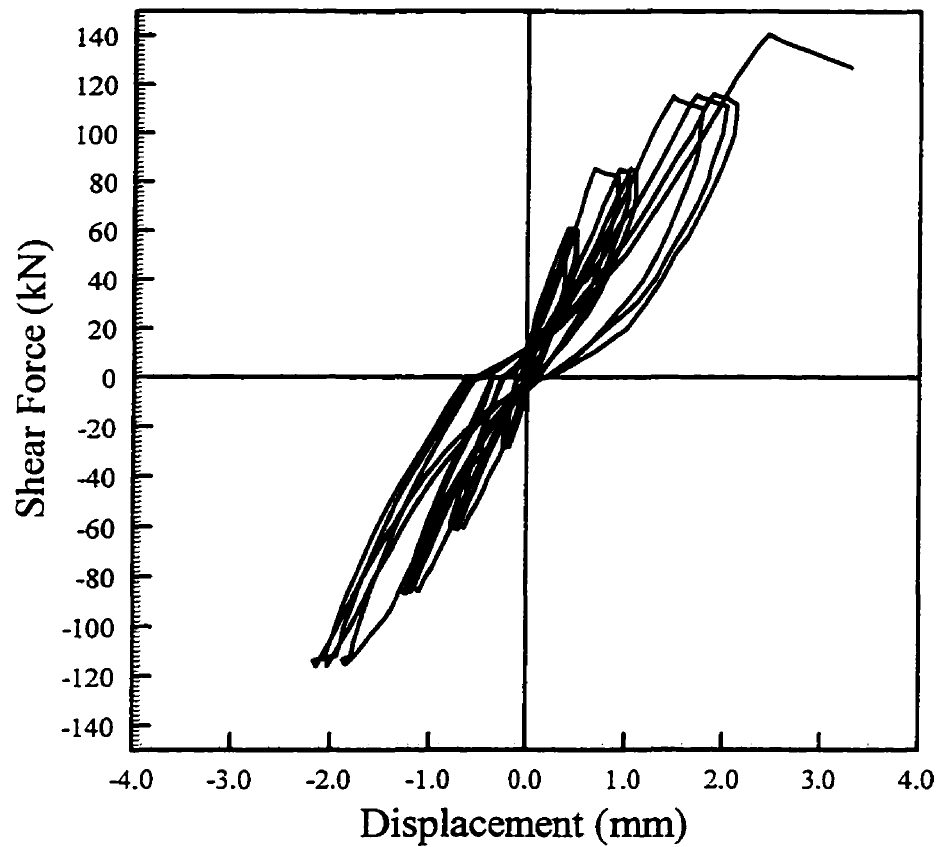
Cycles 10-12, $V=115$ $T=36$ kN The reinforcing steel anchor yielded at Location 1 during Cycle 10A (Figures 3.27 and 3.28). The measured displacements at the yielding stage were 1.6 mm and 0.81 mm for horizontal and axial loads, respectively. Figure 3.29(a) shows a photograph of the specimen at the yielding stage, indicating the density of cracking over the specimen. During the two other cycles at this load level (Cycles 11, 12) the displacement did not increase significantly

Cycles 13, $V=140$ $T=29$ kN This stage of the loading was implemented using displacement control with the maximum displacement equal to 2 Δ_y . The measured loads were 140 kN and 29 kN for shear and axial forces, respectively. The specimen failed at this stage of loading. The ultimate horizontal and axial displacements were measured to be 3.3 mm and 1.25 mm, respectively, giving a maximum displacement ductility ratio of 2.0. Due to complete deterioration of the concrete along the reinforcing bar, specially close to the free edge, the connection lost most of its strength. The test was terminated as soon as the load dropped below 70 percent of the ultimate capacity to prevent excessive damage to the instrumentation used. The area under the loops of hysteresis curves increased significantly after yielding took place, indicating the energy dissipating capability of the connection during the development of yielding (Figures 3.25 and 3.26). The mode of failure was due to splitting of the concrete along the tensile leg (splitting

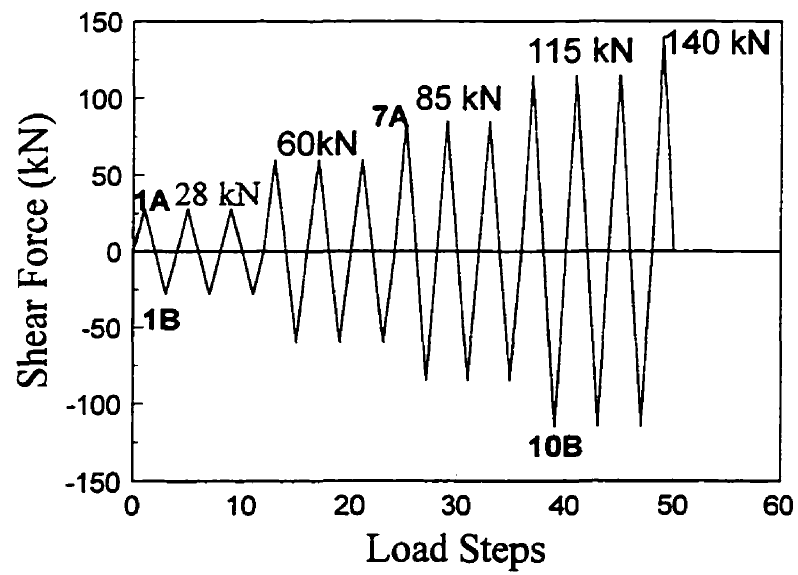
bond failure) along with the breaking away of the concrete close to the free edge, distressed by combined shear force and tensile axial force. Figure 3.29(b) shows the photograph of the specimen after testing, which explains the failure mode of the specimen.

The distribution of the strains along the reinforcing bar prior to failure indicates that more than one third of the reinforcing bar in the tensile zone had yielded (Figure 3.30). These distribution characteristics reveal that no yielding was developed in the compression zone, i.e. at locations 4, 5, or 6.

Generally, the displacement results for axial response show some shift of displacement at zero load. Most of this shift was due to application of shear load and imperfect closing of cracks. A comparison of the shear hysteresis response with that for Specimen AS, shows that it is very similar because the applied cyclic axial load was small and did not have a significant influence on the shear response. The large loops in the positive loading direction (combined shear and tensile loads) and smaller loops in negative direction due to compression axial load were observed. A different load deflection pattern was noted for axial response compared with the response of the Specimens AST1, AST2 and AT. The most important difference was related to the pattern of the load-deflection curve in the negative direction which for Specimen AST3 is larger due to application of higher shear forces and smaller compression axial loads. The displacement in the positive direction had slightly increased compared with that for the Specimen AS (pure cyclic shear), and decreased in comparison with those for Specimens AST1 and AST2. The crack pattern was similar to those for Specimens AST1 and AST2, and different from those for the Specimens AS and AT.

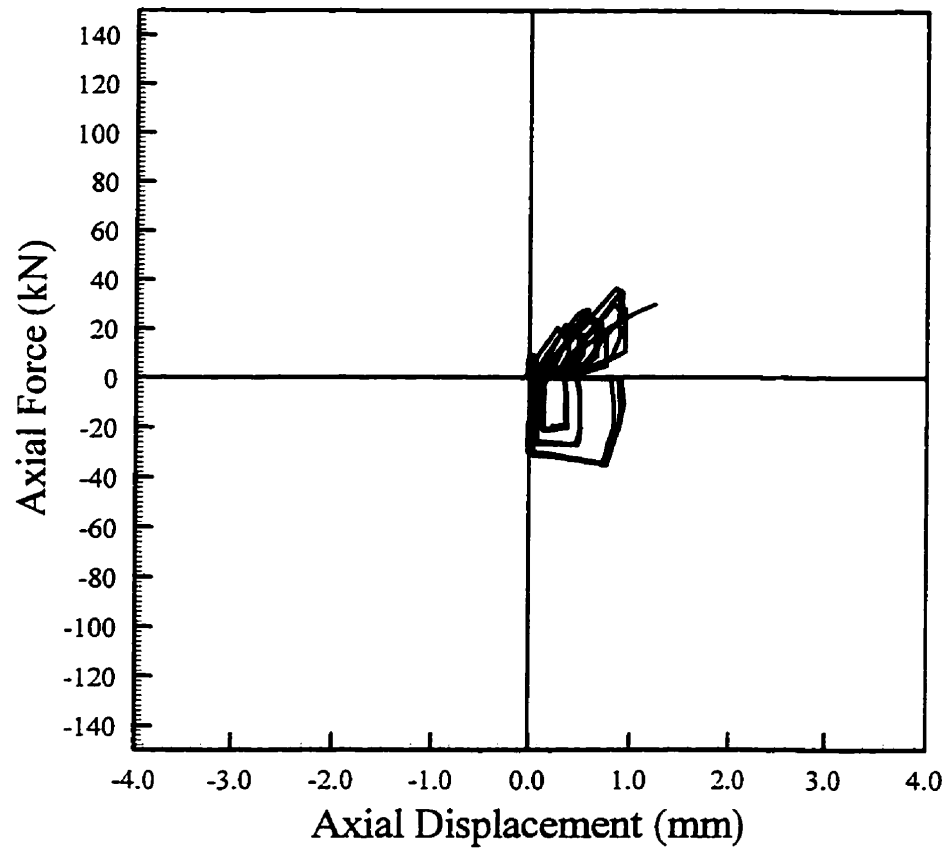


(a) Hysteresis response

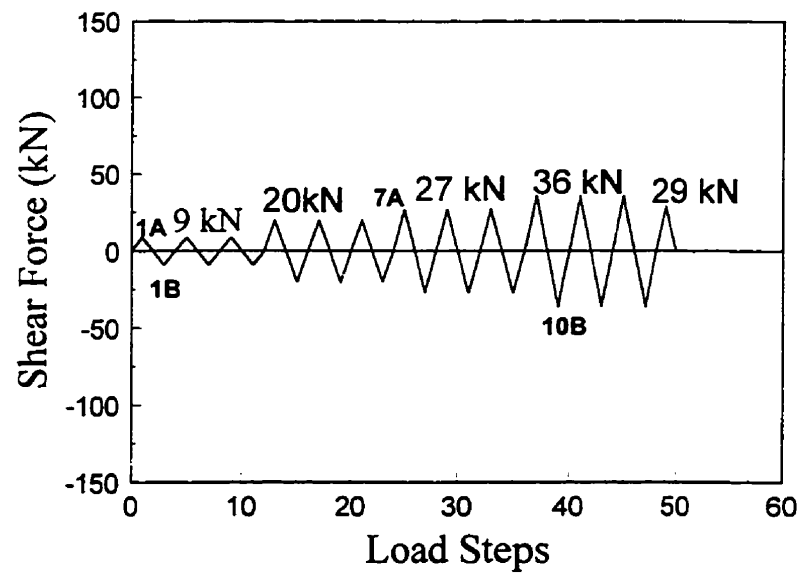


(b) Loading history

Figure 3.25 Hysteresis shear response and loading history for Specimen AST3

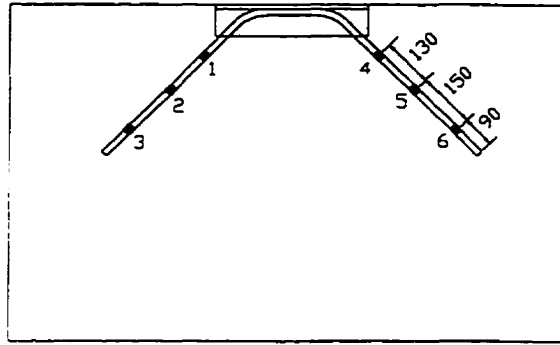


(a) Hysteresis response

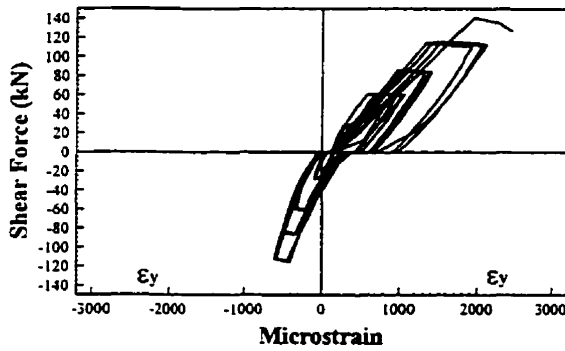


(b) Loading history

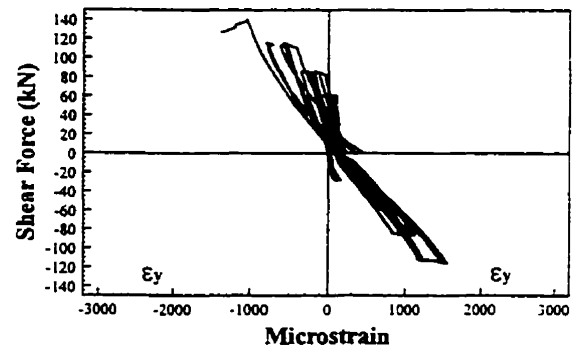
Figure 3.26 Hysteresis axial response and loading history for Specimen AST3



(a) Location of strain gauges

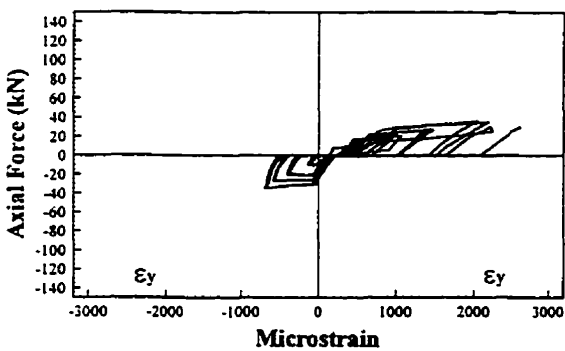


(b) Strains at Location 1, Shear response

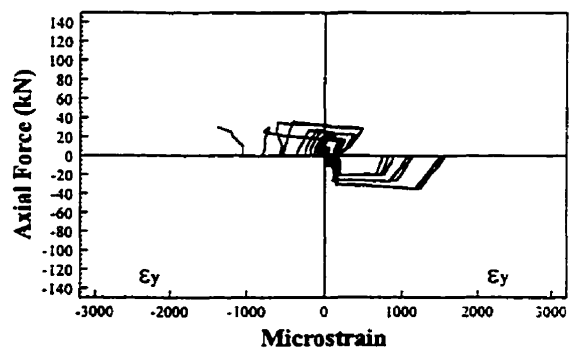


(c) Strains at Location 4, Shear response

Figure 3.27 Strains along length of anchor in Specimen AST3 (Shear response)

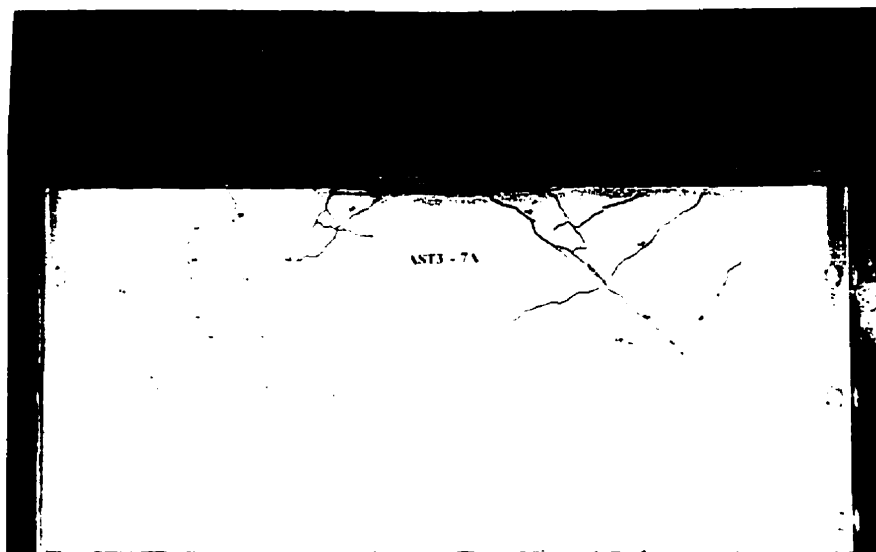


(b) Strains at Location 1, Axial response

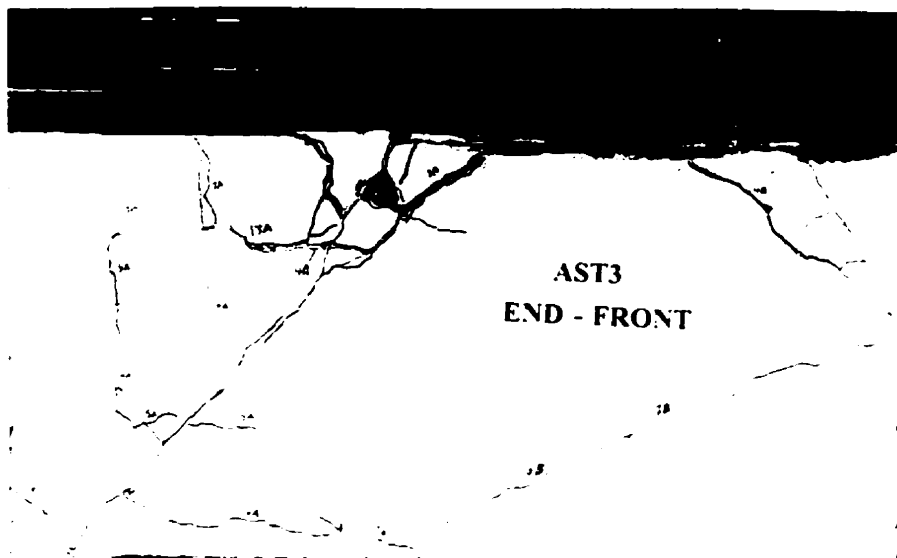


(c) Strains at Location 4, Axial response

Figure 3.28 Strains along length of anchor in Specimen AST3 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}}=115 \text{ kN}$, $P_{\text{axial}}=36 \text{ kN}$



(b) Connection after testing

Figure 3.29 Photographs of Specimen AST3

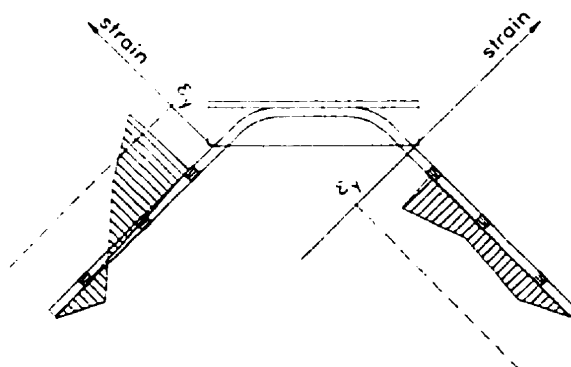


Figure 3.30 Strain distribution along the length of bar prior to failure, Specimen AST3

3.2.7 Interaction Diagram for Series A

During an earthquake, connections in the precast floor system are subjected to combined cyclic shear and axial loads. Safe design of a floor system requires an accurate knowledge of the load capacity of the connection under various combinations of shear and axial forces induced during seismic activity. To develop this knowledge, a detailed investigation is needed to understand the interaction behaviour and to derive the interaction diagram and the associated equation between the shear and axial load.

In the previous sections (Sections 3.2.2 to 3.2.6), the general behaviour and the ultimate capacities of the specimens in Series A tested under different combinations of cyclic shear and axial loadings were discussed. Reversed cyclic tests conducted on Series A specimens were performed with pure shear, pure axial and the combination of axial to shear with T/V ratios of 0.7 (AST1), 1.0 (AST2) and 0.3 (AST3). The results are summarized in Table 3.6. The maximum cyclic shear, expressed as V/V_o are plotted versus the axial cyclic ratios, T/T_o , in Figure 3.31. For design purposes a so-called "experimental interaction equation" has been fitted to the experimental results; given by:

$$\left(\frac{V}{V_o}\right) + \left(\frac{T}{T_o}\right)^{1.75} \leq 1.0 \quad (3.1)$$

where V is the applied shear force, V_o is the maximum force that the connection can resist in pure cyclic shear, T is the applied axial force, and T_o is the maximum force that the connection can resistance in pure axial loads. The material resistance factors must be included so the diagram gives the interaction relation for design, see Chapter 7.

It can be noted that the cyclic shearing strength decreases when a cyclic axial force is applied, even for small values of the axial force, because the axial forces are transferred to the reinforcing bar by the face steel angle, which cause tensile stresses in the concrete along the reinforcing bar (bond stress), and, also, stress the concrete at the interface of the steel angle, thereby weakening the bond strength when the shear forces are applied. Furthermore, the resulting shear stiffness and the cracking load reduce significantly due to the application of the axial forces. The effect of the shearing forces on the axial response is similar, because of the same phenomena, i.e., the cyclic axial strength decreases when an cyclic shear force is applied.

Table 3.6 Summary of interaction test results for Series A

Test type	Ratio of axial to shear (T/V)	Ultimate strength (kN)		Ratios	
		Shear (kN)	Axial (kN)	V/V _o	T/T _o
AS	-	150	-	1.00	0.0
AST1	0.7	88	62	0.587	0.590
AST2	1.0	75	75	0.500	0.714
AST3	0.3	126	30	0.840	0.286
AT	-	-	105	0.0	1.0

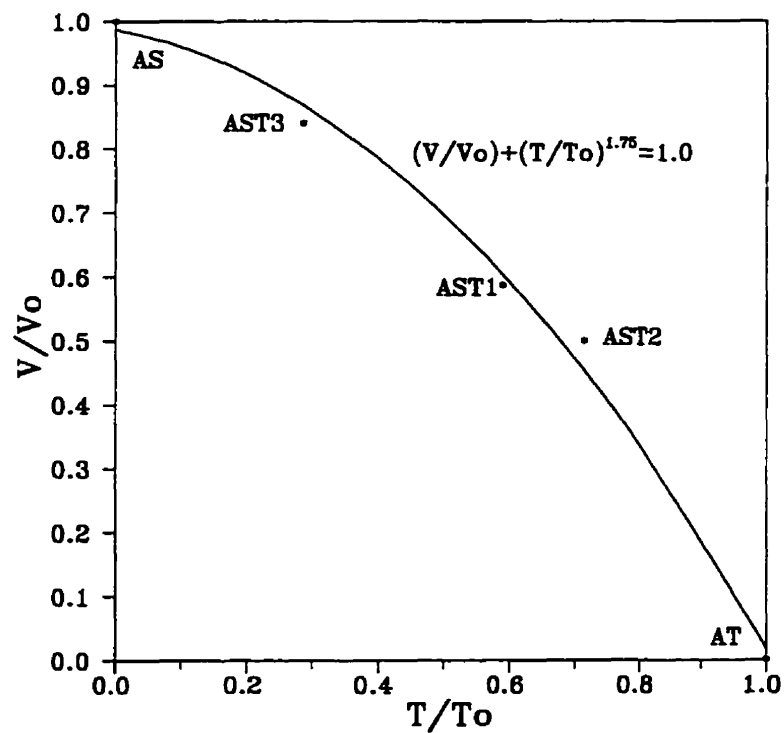


Figure 3.31 Normalized interaction diagram for Series A

3.3 Connection Type Ap: specimen with bent steel bar anchor welded to steel plate

This connection was identical to the Connection Type A, except that the anchor bar was welded to 200 x 50 x 10 mm steel plate instead of the steel angle. This was designed to investigate the response characteristics of the connection with a steel plate and to examine the mechanism of the possible fracture failure of the weld between the steel plate and steel bar.

3.3.1 Specimen ApS

The Specimen ApS was subjected to reversed cyclic shear only, as summarized in Table 3.7. The loading history and the hysteresis response of the specimen are presented in Figure 3.32. Figure 3.33 shows the shear force versus the strains along the anchor bars. The distribution of strains with the distance along the length of reinforcing bar prior to failure is shown in Figure 3.34. The photographs of the specimen at the yield stage and after failure are shown in Figures 3.35(a) and 3.35(b), respectively. The following observations were made during the testing and from experimental results.

Cycles 1-3, 45 kN No crack was visible. The load-deflection response remained linear elastic up to end of this load stag.

Cycles 4-6, 85 kN The first crack appeared along the tensile reinforcing leg during Cycle 4A at a load of 65 kN. This caused a change of the slope in the load-deflection response (Figure 4.5), however, during the following cycles of this load stage, linear responses were noted accompanied with very small area under the hysteresis loops, see Figure 3.32. The maximum crack width along the bar anchor was 0.2 mm. No notifiable energy dissipated up to end of this stage.

Cycles 7-9, 145 kN The anchor bar yielded at Location 1 during cycle 7A at a load value 132 kN (Figure 3.33) resulting in a change in the stiffness of the connection and an increase in the displacement to $\Delta_y = 1.4$ mm (Figure 3.32). The splitting crack widened and extended along the entire length of tensile bar leg, while diagonal cracks formed in tension zone [see Figure 3.35(a)]. The maximum crack widths were measured to be 0.4

mm and 0.25 mm for splitting and diagonal cracks, respectively. During the following cycles at this load stage, the area under the loops did not increase significantly (Figure 3.32), resulting in an insignificant energy dissipation.

Cycle 10, 153 kN This stage involved the use of displacement control with a maximum deflection of $\Delta = 2\Delta_y = 2.9$ mm. The anchor bar yielded at Location 2 (Figure 3.33), indicating that yielding developed along the bar anchor. This caused a significant decrease in the slope of the load -deflection curve showing a reduction in the stiffness of the connection. The cracks were distributed all over the specimen and the concrete around the steel plate spalled off. During the following cycles, due to development of yielding and crack propagation, the area under the hysteresis loops increased. The crack width at the peak of the cycle were 1 mm and 0.8 mm for splitting and diagonal cracks, respectively.

Cycle 11, 120 kN This stage was implemented using displacement control with $\Delta = 4\Delta_y = 5.5$ mm. The maximum load which could be applied was reduced to 120 kN, which was due to spalling of concrete around part of 45° bent, rotation and bending of bar in this region, and breaking away of the concrete around the steel plate. All these events resulted in beginning of fracture of the weld at the steel plate-anchor bar interface. The area under the hysteresis loops increased significantly in spite of the decrease in the load carrying capacity. This indicates incapability of the connection to sustain the maximum load.

Cycle 12, 95 kN Finally, the specimen was subjected to a displacement of $\Delta = 5.5\Delta_y = 7.7$ mm ($P = 123$ kN). The stiffness and strength kept on decreasing through repeating load cycles, which was due to gradually increasing fracture of the weld. The specimen failed during Cycle 12B. The area under the loops increased very significantly.

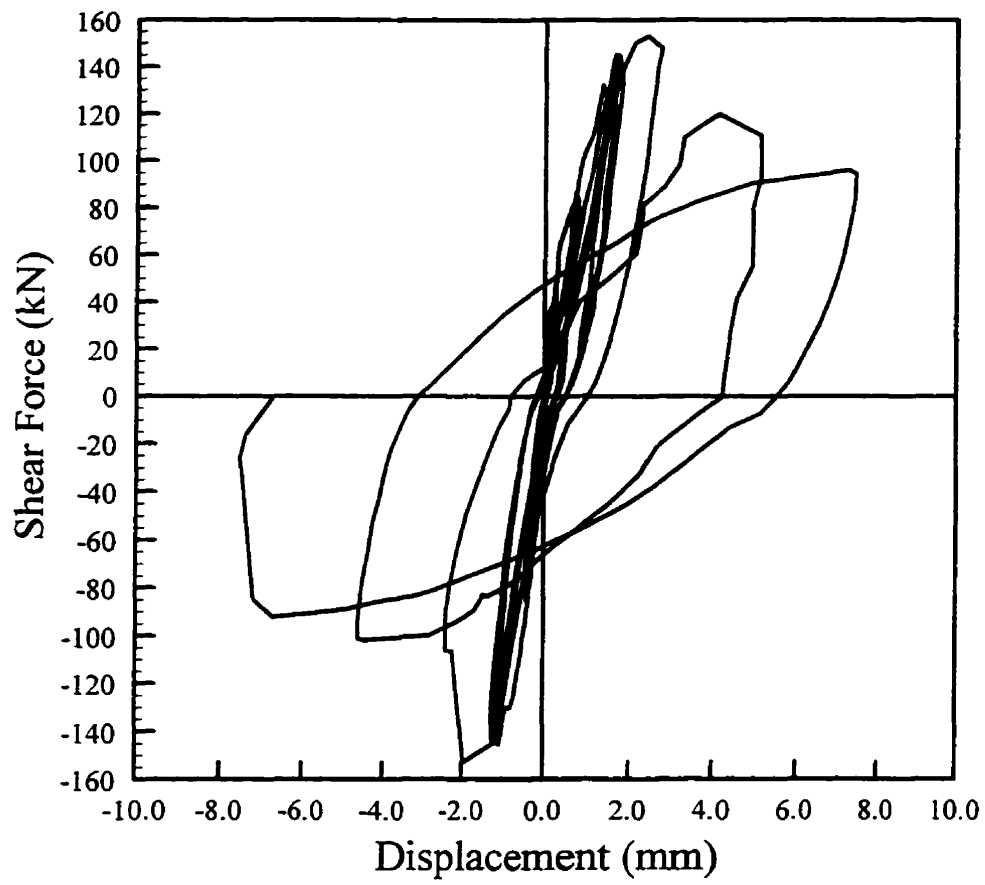
The variation of the axial strains along the anchor bar legs at the ultimate load shows that more than four-fifth of the tension leg of anchor had yielded (see Figure 3.34). The 153 kN capacity was attained while the final failure mode was complete fracture of the weld at the steel plate- reinforcing anchor interface and crushing-spalling of the concrete around the steel plate [see Figure 3.35(b)]. The hysteresis response consisted of

large loops beyond the yielding of reinforcing bar, specially during the two last cycles. The ultimate displacement prior to failure was 7.7 mm, giving a displacement ductility ratio of 5.5.

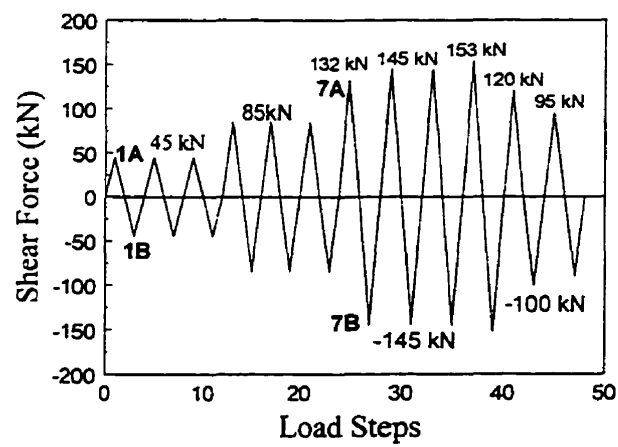
Table 3.7 Summary of Loading History for Specimen ApS

Cycle	Load Stages	Horiz. Displacement (mm)*	Shear Force (kN)
Elastic	1A - 3B	+0.1 , -0.1	±45
Cracked	4A - 6B	+1.1 , -1.1	±85
±1 Δ_y	7A - 9B	+1.9 , -1.7	±145
±2 Δ_y	10A - 10B	+2.9 , -2.0	±153
±4 Δ_y	11A - 11B	+5.5 , -4.6	±120
Failure, +5.5 Δ_y	12A - 12B	+7.7, -7.1	+95

* Maximum displacement at the peak load of the last cycle

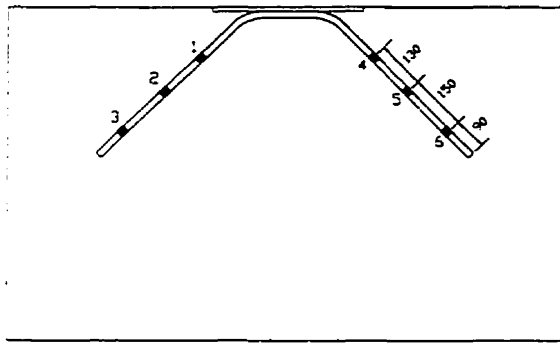


(a) Hysteresis response

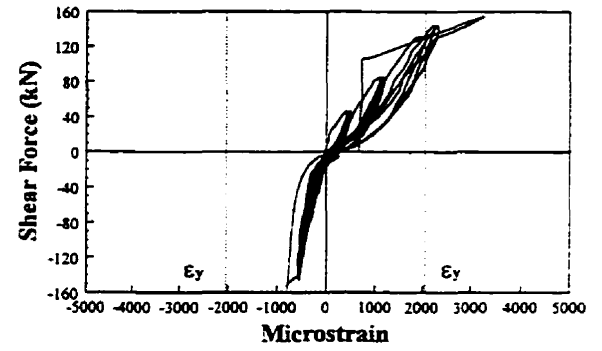


(b) Loading history

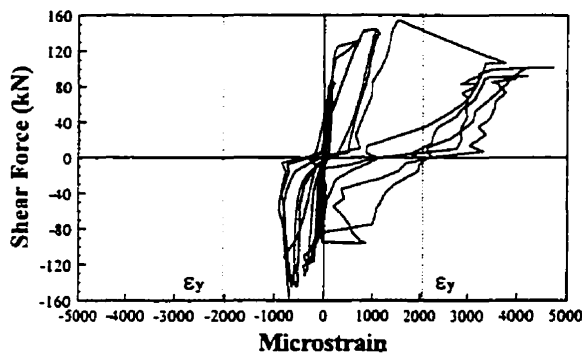
Figure 3.32 Hysteresis response and loading history of Specimen ApS



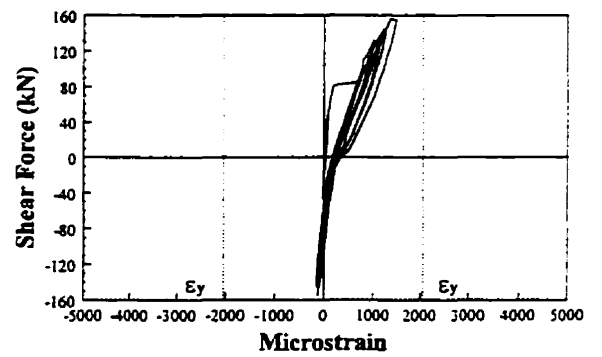
(a) Location of strain gauges



(b) Strain at Location 1



(c) Strain at Location 2



(d) Strain at Location 3

Figure 3.33 Measured strains along the length of anchor for Specimen ApS

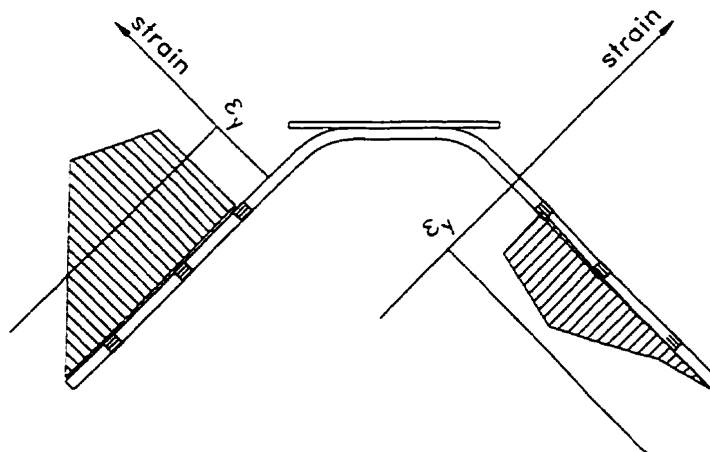
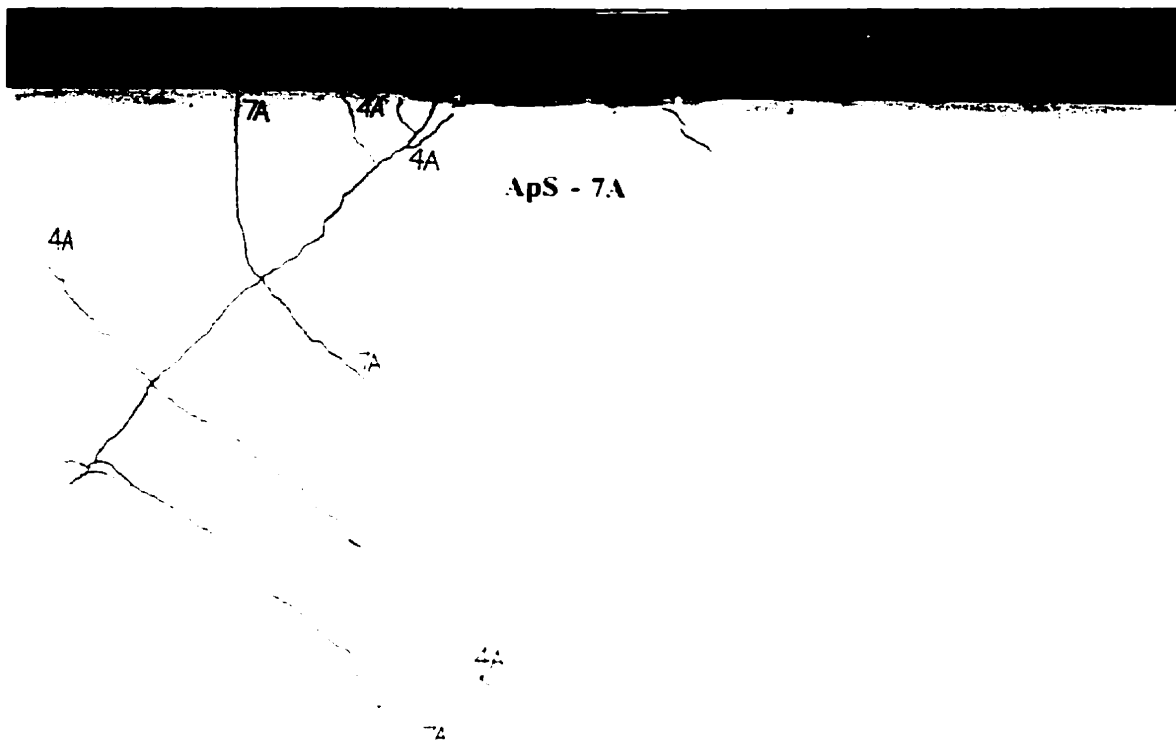
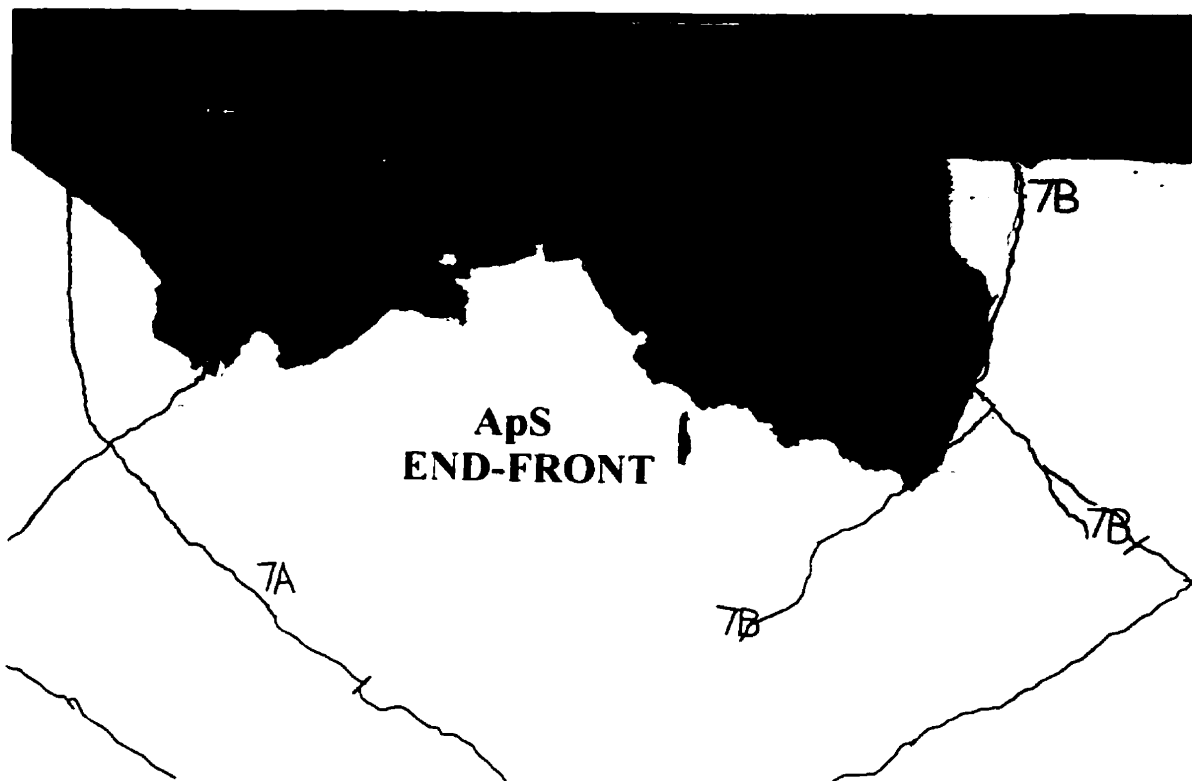


Figure 3.34 Strain distribution along the length of the bar prior to failure, Specimen ApS



(a) Connection at first yield, Δ_y , $P=132$ kN



(b) Connection after testing

Figure 3.35 Photographs of Specimen ApS

3.4 Test Series B: Connection with bent steel bar and 90° hook welded to an angle

This type of connection consisted of No 15 deformed bar bent at 45° with 90° standard hook at the end welded to 10 mm thick steel angle (see Figure 2.15). A comprehensive description of connection can be found in Section 2.5. All calculations and determination of ultimate strength were performed following the same method as described for Test Series A. Six identical specimens were tested under different load combinations - one under monotonically increasing shear (Specimen BMS) and the rest (Specimens BS, BT, BST1, BST2 and BST3) subjected to combined cyclic shear and axial loads with different values of the T/V ratio.

3.4.1 Specimen BMS

Figure 3.36 shows the shear force - horizontal displacement curve for the specimen. The load versus measured strains along the length of bar can be seen in Figure 3.37. The profile of the strains prior to failure versus distance along the length of bar is shown in Figure 3.38. Figures 3.39(a) and 3.39(b) show photographs of connection at the yield stage and after failure, respectively. The following general behaviour were noted during testing and analysis of the experimental results.

Stage 1 95 kN Up to this stage load-deflection remained linear (Figure 3.36). The first crack appeared along the bar with a maximum crack width of 0.4 mm. The slope of the load-deflection curve decreased after the initiation of the first crack.

Stage 2 165 kN The splitting crack extended along the bar, three diagonal cracks appeared perpendicular to the splitting one. The reinforcing bar at Location 1 yielded (Figure 3.37). The lateral displacement at yield was $\Delta_y = 1.8$ mm with a measured crack width of 1.0 mm. A horizontal crack appeared at the back of hook [Figure 3.38(a)].

Stage 3 215 kN A crack was observed along the compression leg due to large bond stress and dowel action. The measured displacement was 3.6 mm, caused by the $(2\Delta_y)$, while the maximum crack width in tension zone was 2.5 mm.

Stage 4 242 kN The reinforcing bar yielded at Location 2. The slope of the load-deflection curve kept on decreasing up to the ultimate load of 242 kN. The load decreased to 237 kN and the mode of failure was due to splitting and spalling of the concrete along

the tension leg and due to crushing and spalling of the concrete in front of the angle [Figure 3.38(b)]. The maximum displacement was 6.8 mm giving a ductility ratio of 3.8.

The ultimate load attained was 242 kN. The load-displacement response showed flexible behaviour after the yielding of reinforcing bar. It was noted that the cracking of the concrete at the yield level was significant. The variation of strains along the length of the reinforcing bar at the ultimate load (Figure 3.37) indicates that more than one half of the tension reinforcing bar had yielded. No yielding was noted in compression leg.

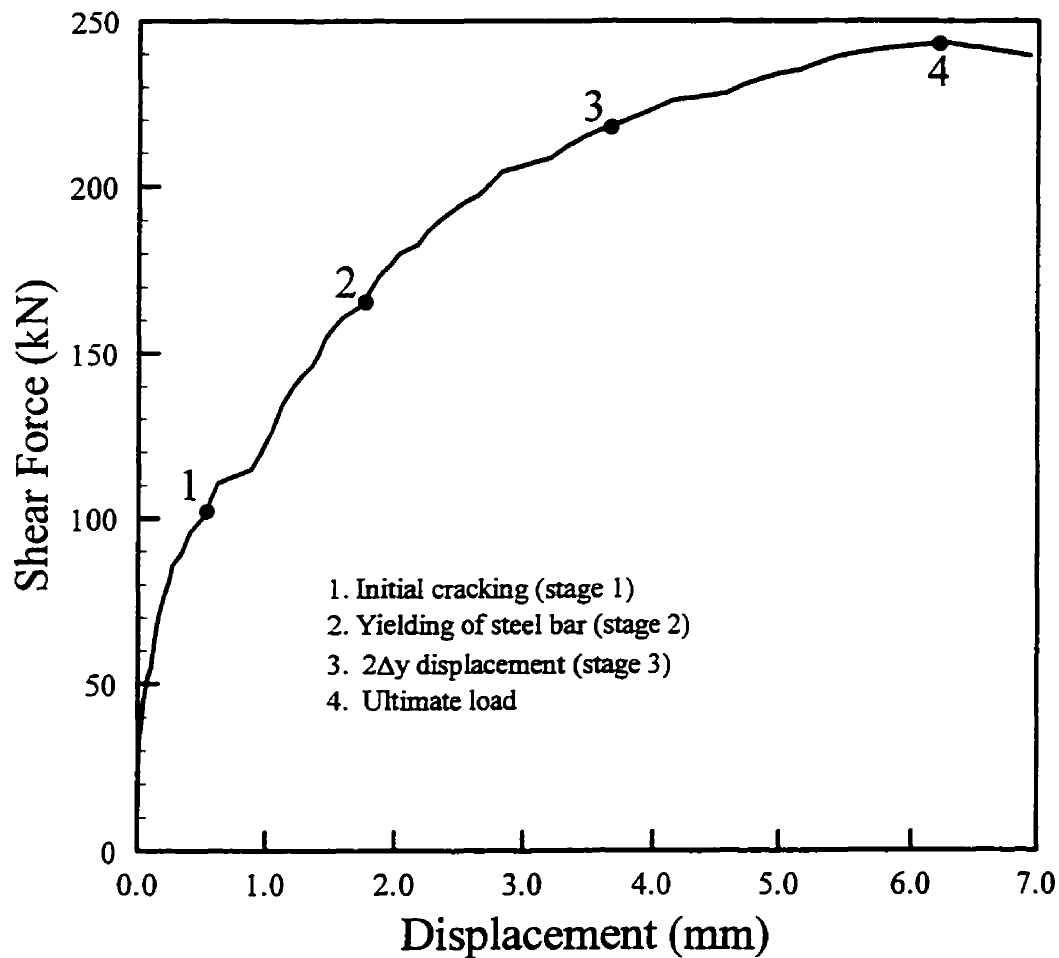
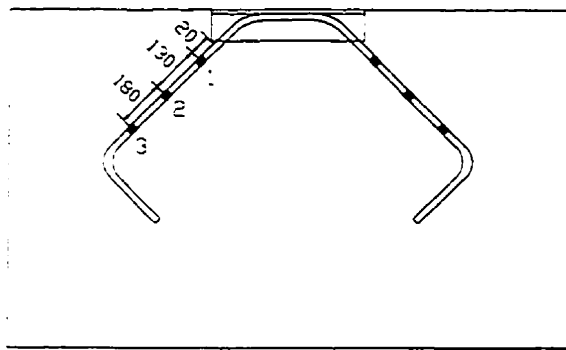
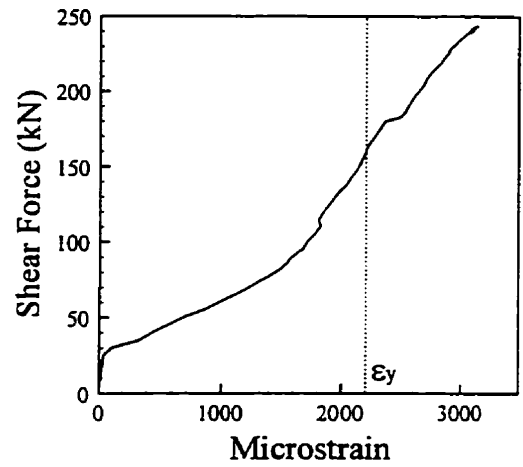


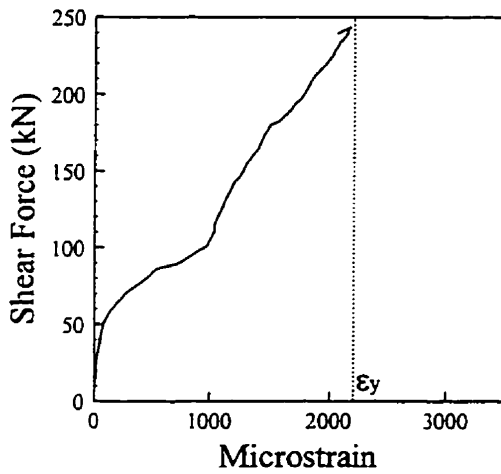
Figure 3.36 Shear force vs. horizontal curve of Specimen BMS



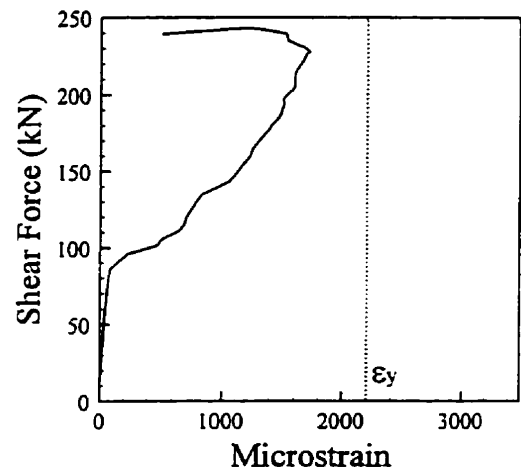
(a) Location of strain gauges



(b) Strains at Location 1



(c) Strains at Location 2



(d) Strains at Location 3

Figure 3.37 Measured strains along the length of anchor for Specimen BMS

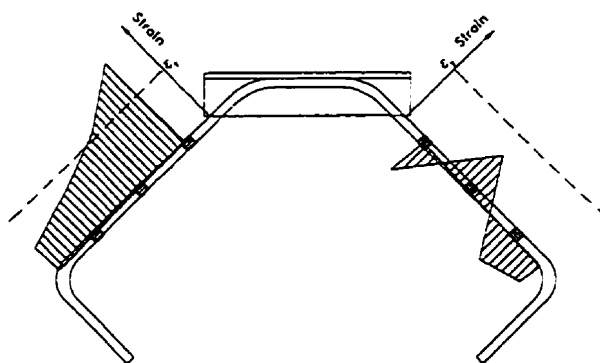
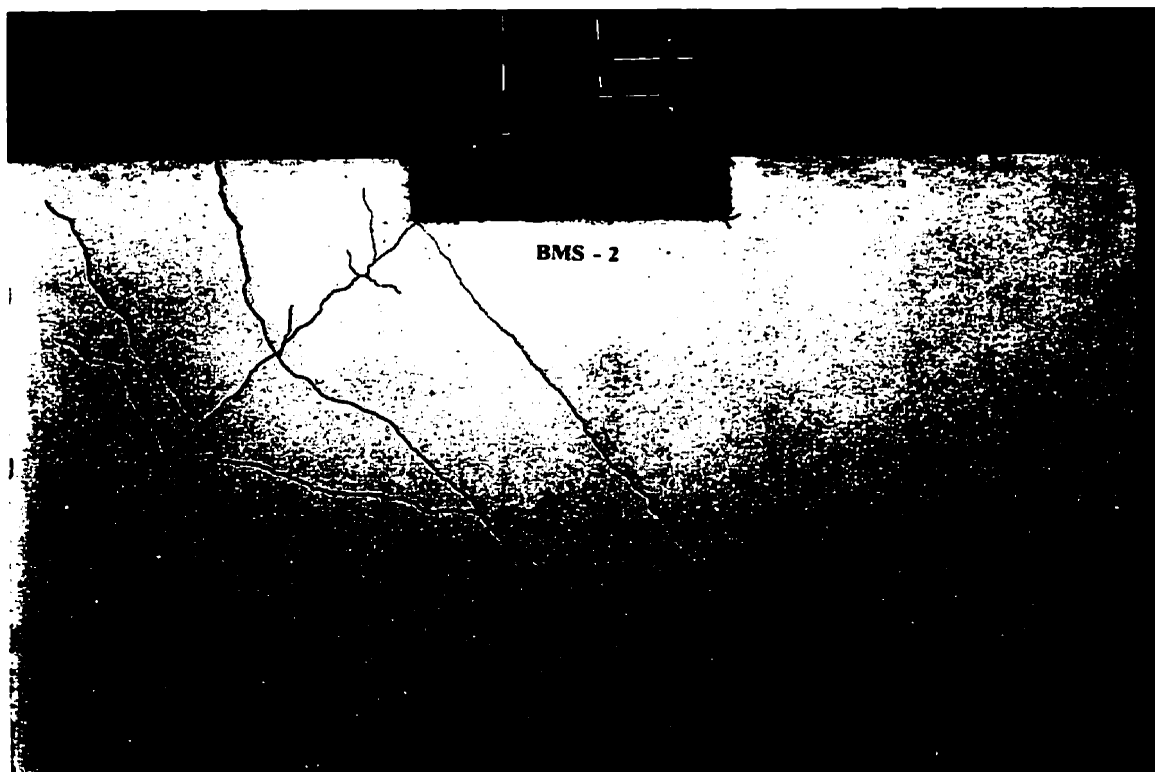
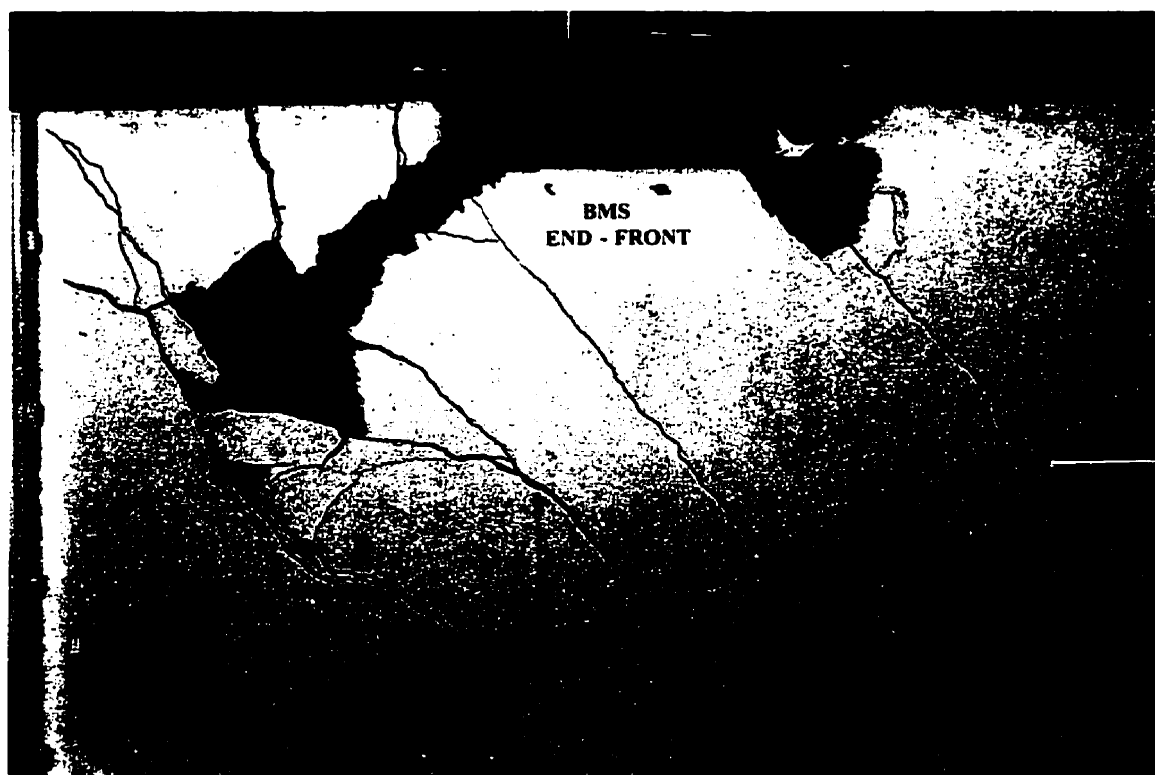


Figure 3.38 Strain distribution along the length of bar prior to failure of Specimen BMS



(a) Connection at first yield, Δ_c , $P=165$ kN (Stage 2)



(b) Connection after failure

Figure 3.39 Photographs of Specimen BMS

3.4.2 Specimen BS

The Specimen BS from Series B was loaded under reversed cyclic shear only, as summarized in Table 3.8. Table 3.8 and Figure 3.40 show the loading history and hysteresis response of the specimen. The shear force versus strains along the anchor are presented in Figure 3.41. The distribution of the strains with the distance along the length of reinforcing bar prior to failure is shown in Figure 3.42. The photographs of the specimen at the yield level and after failure are shown in Figures 3.43(a) and 3.43(b), respectively. The following observations of behaviour were made during the testing and from analysis of the experimental results:

Cycles 1-3, 66 kN No cracks were visible. The behaviour was basically linear elastic with no change in the slope of the load deflection curve.

Cycles 4-6, 100 kN At a load of 82 kN, small cracks along the reinforcing bar (splitting) and perpendicular to that (diagonal) were observed. These cracks propagated during the following cycles at this load level. The crack patterns were symmetrical. Nonlinear response started due to cracking (Figure 3.40). The dissipated energy was not noticeable up to this stage.

Cycles 7-9, 150 kN The anchor bar yielded at Location 1 (Figure 3.41) resulting in an increase in the displacement to $\Delta_y = 1.6$ mm. The splitting crack extended and opened more ($w_{max} = 0.5$ mm) and the cracks at the location of hook opened to 0.6 mm [Figure 3.43(a)]. The other anchor leg yielded at Location 4 when loaded in the opposite direction (B-cycle) and the splitting cracks extended along the entire length of both anchor legs. The loops were pinched, resulting in an insignificant energy dissipation.

Cycles 10-11, 200 kN This loading stage was implemented using displacement control with a maximum displacement of $2\Delta_y = 3.2$ mm. The anchor bar yielded at Location 2 indicated development of yielding along the bar anchor. The crack density extended all over the specimen with a symmetric pattern and the area under the loops increased due to developing of yielding and crack propagation resulted in more energy dissipated by connection. During Cycle 11B (-200 kN), the anchor bar at Location 5 yielded and the concrete at the front of the hook started to crush where some concrete at the front of the angle spalled off. During Cycle 12B, the connection failed at a load of -184 kN and the

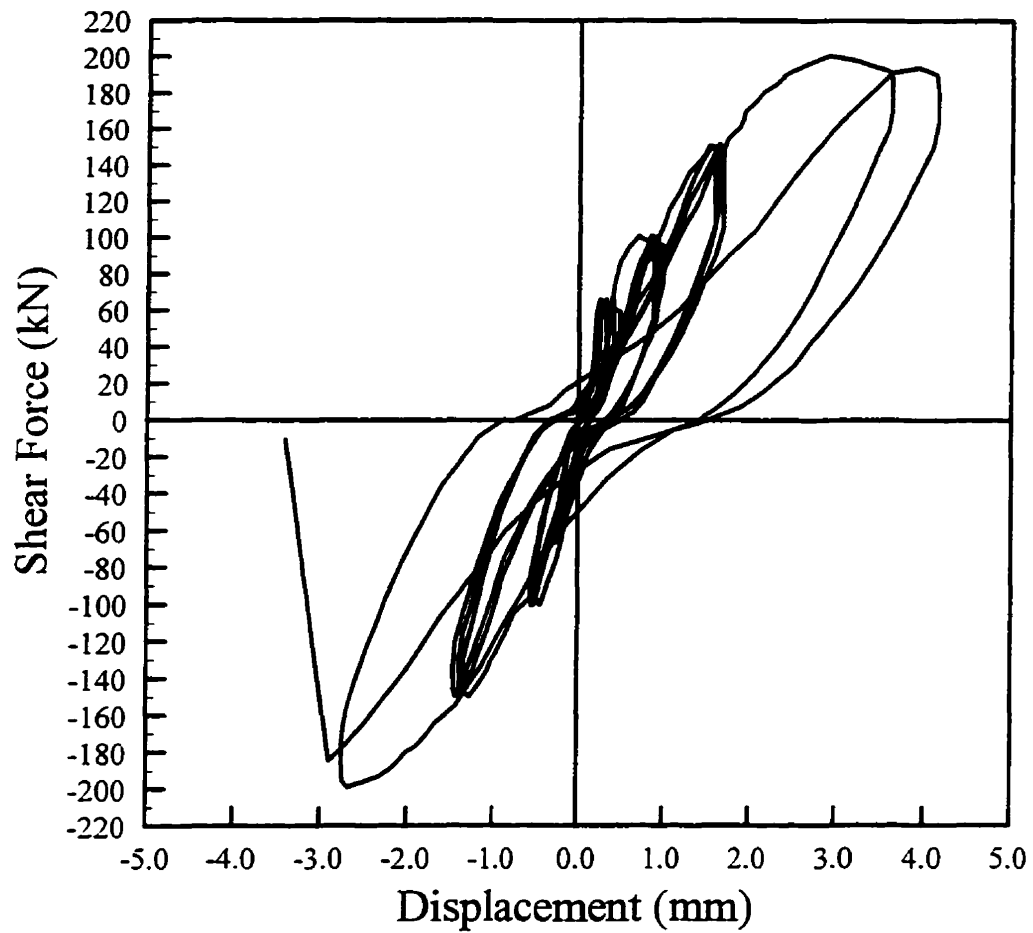
test was stopped.

The profile of strains along the anchor bar legs at the ultimate load shows that two-third of the straight part of the anchor had yielded (Figure 3.42). The ultimate capacity achieved was 200 kN, while the failure mode was due to splitting and spalling of concrete along the anchor bar legs along with the crushing and spalling of the concrete in front of the 90° hook [Figure 3.43(b)]. The hysteresis response shows large loops beyond the yielding of reinforcing bar, which indicates the capability of energy dissipation after yielding of anchor bar. The maximum measured displacement during the test was 4.2 mm, giving a displacement ductility ratio of 2.6.

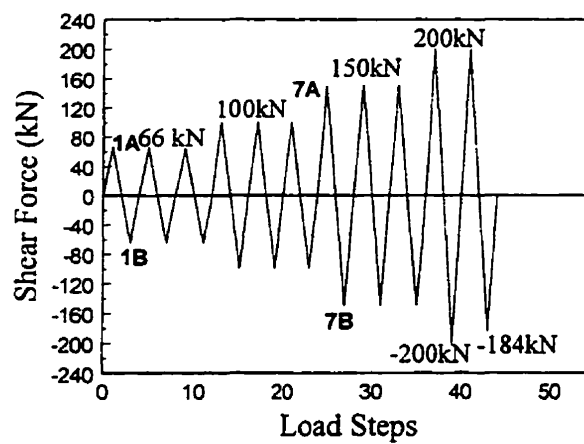
Table 3.8 Summary of loading history for Specimen BS

Cycle	Load Stages	Horiz. Displacement (mm)*	Shear Force (kN)
Elastic	1A - 3B	+0.4 , -0.3	±66
Cracked	4A - 6B	+0.9 , -0.6	±100
±1 Δ_y	7A - 9B	+1.7 , -1.5	±150
±2 Δ_y	10A - 10B	+3.2 , -2.8	±200
Failure, 2.6 Δ_y	11A - 11B	+4.2 , -3.4	+194 , -184

* Maximum displacement at the peak load of the last cycle

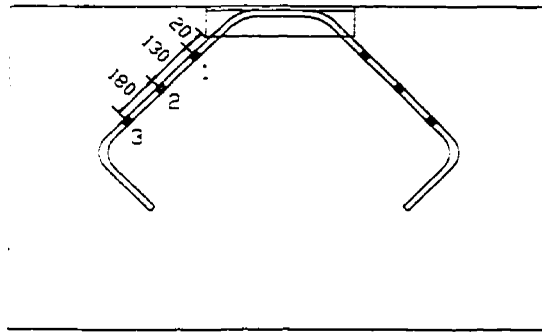


(a) Hysteresis response

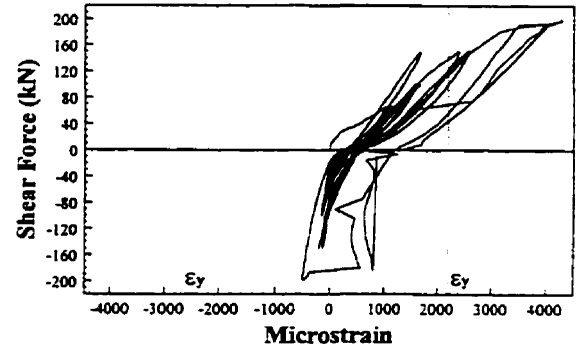


(b) Loading history

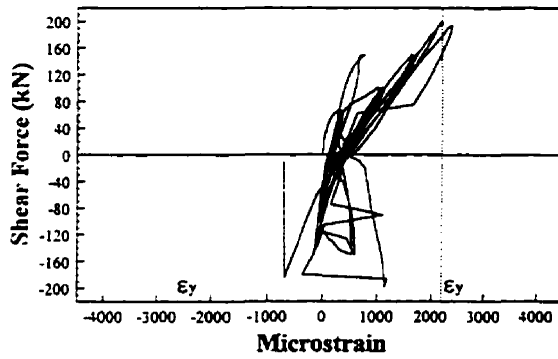
Figure 3.40 Hysteresis response and loading history for Specimen BS



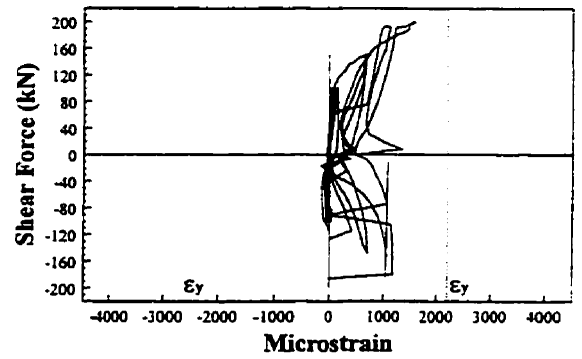
(a) Location of strain gauges



(b) Strains at Location 1



(c) Strains at Location 2



(d) Strains at Location 3

Figure 3.41 Measured strains along the length of anchor for Specimen BS

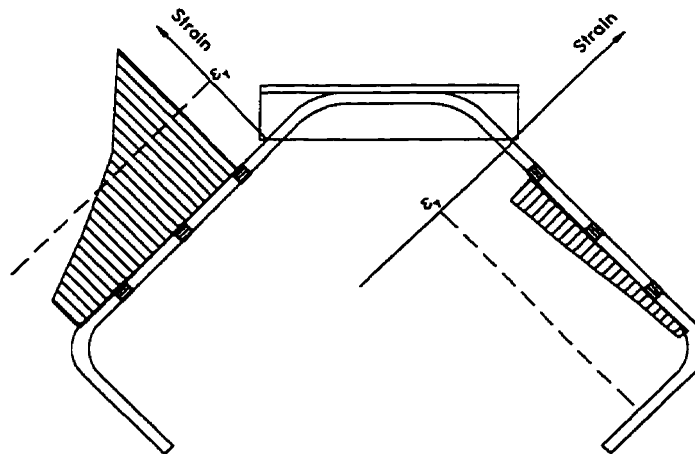
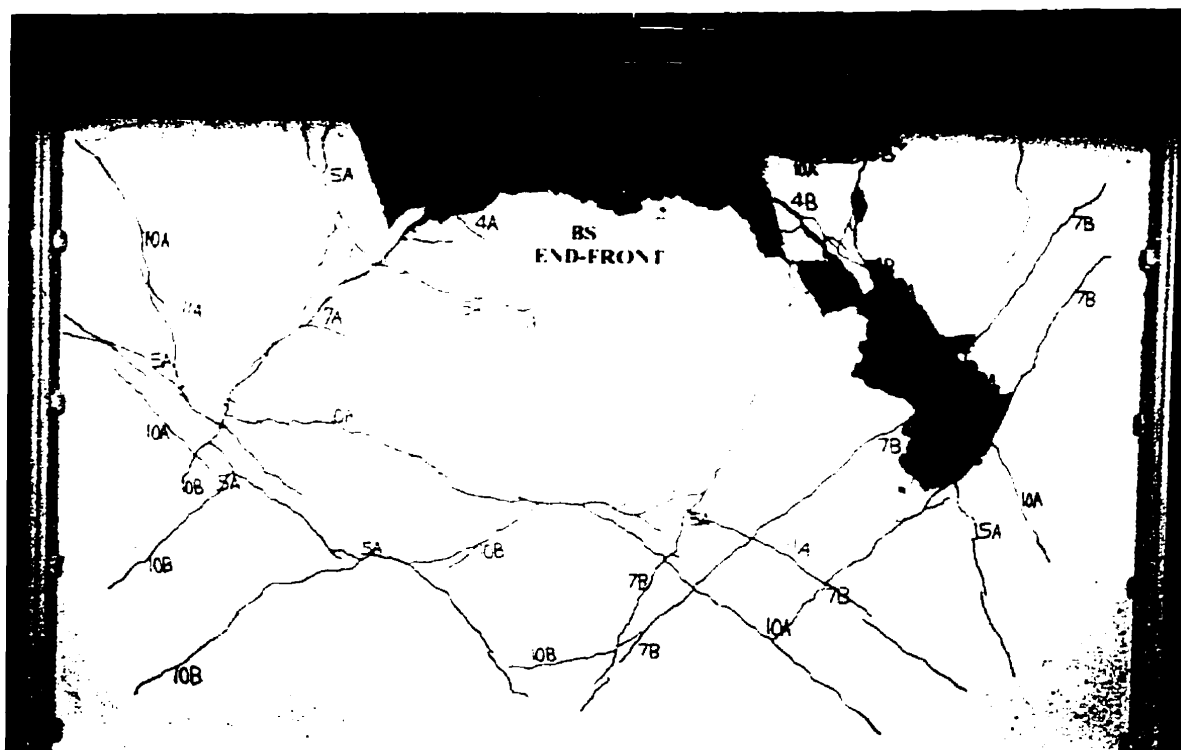


Figure 3.42 Strain distribution along the length of bar prior to failure of Specimen BS



(a) Connection at first yield, Δ_y , $P=150$ kN



(b) Connection after failure

Figure 3.43 Photographs of Specimen BS

3.4.3 Specimen BT

The Specimen BT was subjected to reversed cyclic axial loads only. The loading history and the associated displacements are given in Table 3.9. Figure 3.44 shows the hysteresis response and the loading history of the specimen. The applied axial load versus strains at different locations along the reinforcing bar are shown in Figure 3.45. Figure 3.46 illustrates the distribution of axial strain at ultimate load versus the distance along the straight part of anchor bar. The photographs of the connection at the yield stage and after failure are presented in Figures 3.47(a) and 3.47(b), respectively. The following notes were made based on the observation during testing and analysis of the experimental results:

Cycles 1-3, 66 kN No cracking was noted. The response was completely linear elastic resulting in no energy dissipation

Cycles 4-6, 95 kN The first small cracks (splitting) were observed along both legs at a load of 70 kN, resulting in a change of the load-deflection curve slope where nonlinear behaviour started. At the end of this stage, the maximum crack width was 0.1 mm with similar crack pattern for both legs. The displacement was measured to be 1.1 mm. The area under the loops were small.

Cycles 7-9, 115 kN The reinforcing bar anchor yielded at Locations 1 and 4 (Figure 3.45) at a load of 100 kN. The displacement at yield was $\Delta_y = 1.3$ mm. The splitting crack extended further and a diagonal and horizontal cracks appeared at the back of the hook. The photograph of the connection at the yield level is shown in Figure 3.47(a). The area under loops still was small indicating a low-energy dissipation.

Cycles 10-12, 140 kN This stage was implemented using displacement control with a maximum displacement equal to $2\Delta_y = 2.6$ mm. The cracks widened and propagated with the sign of the breaking of the concrete at the corner of the angle. During the following cycle (11A), the reinforcing bar yielded at location 2 and the area under the hysteresis loops increased significantly indicating energy dissipation through the development of yielding. The strains at Location 1 was complex because of disturbed stress area close to the free edge. The displacements during compression loads (negative) did not recover due to imperfect closing of cracks.

Cycles 13-15, 150 kN

This stage was characterized by a maximum displacement of $4\Delta_y$. No more cracks appeared while the splitting and diagonal cracks widened to 1.0 mm and 1.25 mm, respectively. The stiffness of the connection kept on decreasing and the area under the loops increased, resulting in an increase in the energy dissipation through the yield development.

Cycle 16, 150 kN

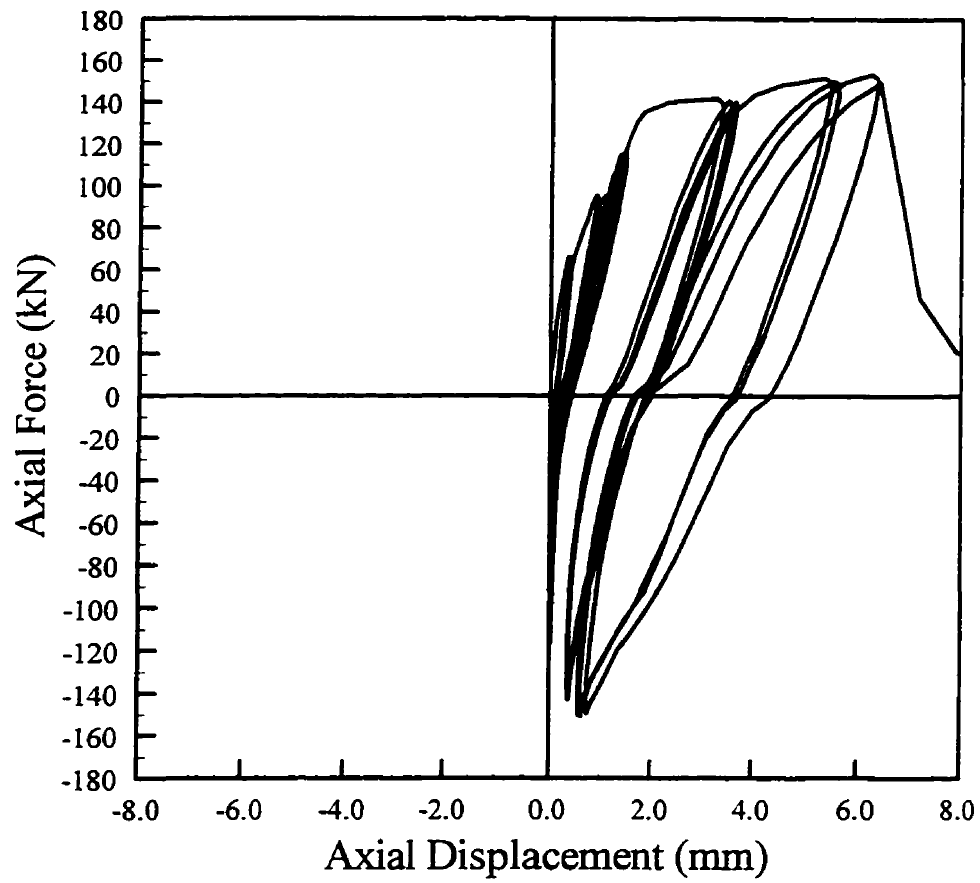
This stage was implemented to control the connection loading for a maximum displacement equal to $6\Delta_y$. The ultimate strength of 150 kN was achieved when the connection failed and the load dropped to about 40 kN. The test was stopped to avoid excessive damage to instrumentation used. The ultimate displacement was 7.9 mm giving a displacement ductility ratio of 6.0.

The strain profile along the bar anchor at ultimate load indicates that almost two-third length of the straight part of the bar had yielded (Figure 3.46). The maximum strength of 150 kN was attained with a mode of failure due to the splitting and spalling of the concrete along the straight part of bar, breaking away of the concrete around the steel angle, and the crushing and spalling of concrete in front of the 90° hooks [Figure 3.47(b)]. The hysteresis response showed large loops in the positive direction which was due to the development of yielding along the reinforcing bar and contribution of the standard hooks. Small loops in the negative direction were due to closing of the cracks and small deformation of the concrete when subjected to compressive stresses.

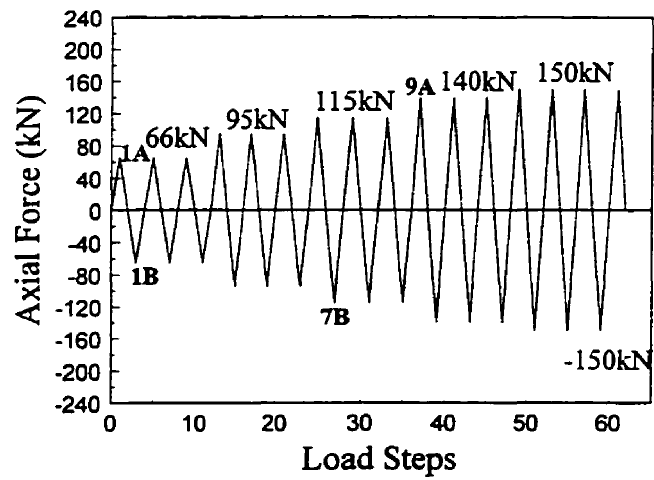
Table 3.9 Summary of loading history for Specimen BT

Cycle	Load Stages	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.4 , +0.1	±66
Cracked	4A - 6B	+1.1 , +0.1	±95
±1 Δ_y	7A - 9B	+1.4 , +0.2	±115
±2 Δ_y	10A - 12B	+3.6 , +0.4	±140
±4 Δ_y	13A - 15B	+6.3 , +0.7	±150
Failure, 6 Δ_y	16A	+7.9	+150

* Maximum displacement at the peak load of the last cycle

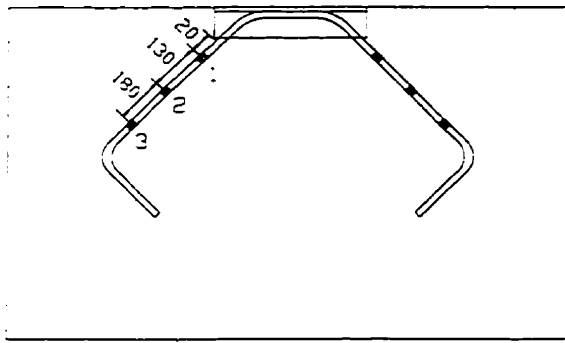


(a) Hysteresis response

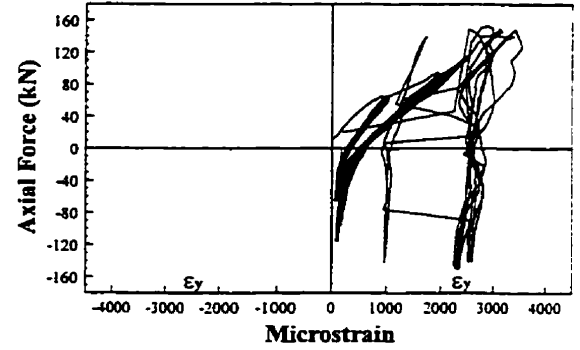


(b) Loading history

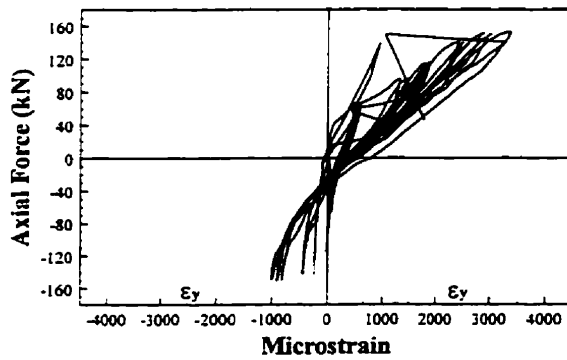
Figure 3.44 Hysteresis response and loading history for Specimen BT



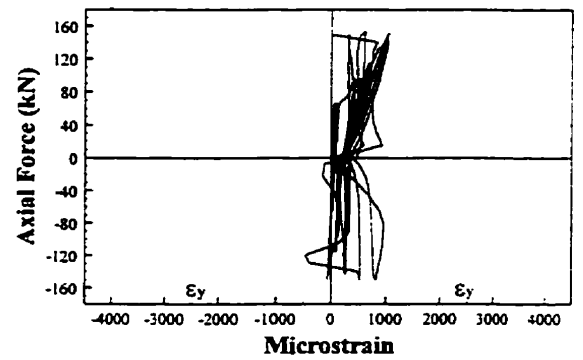
(a) Location of strain gauges



(b) Strains at Location 1



(c) Strains at Location 2



(d) Strains at Location 3

Figure 3.45 Measured strains along the length of anchor for Specimen BT

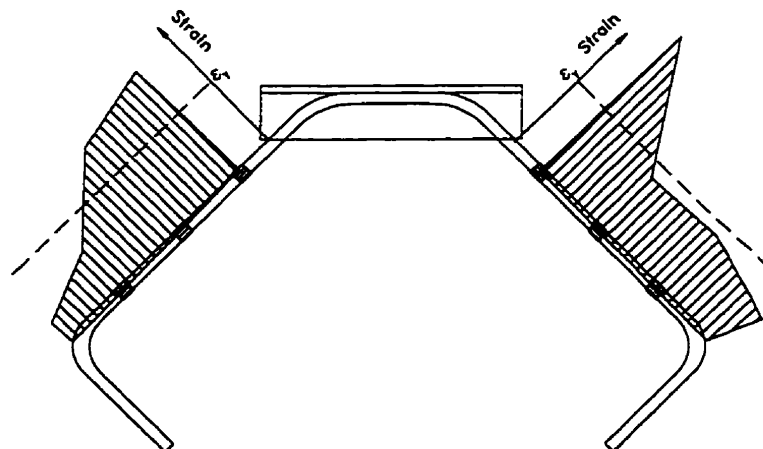
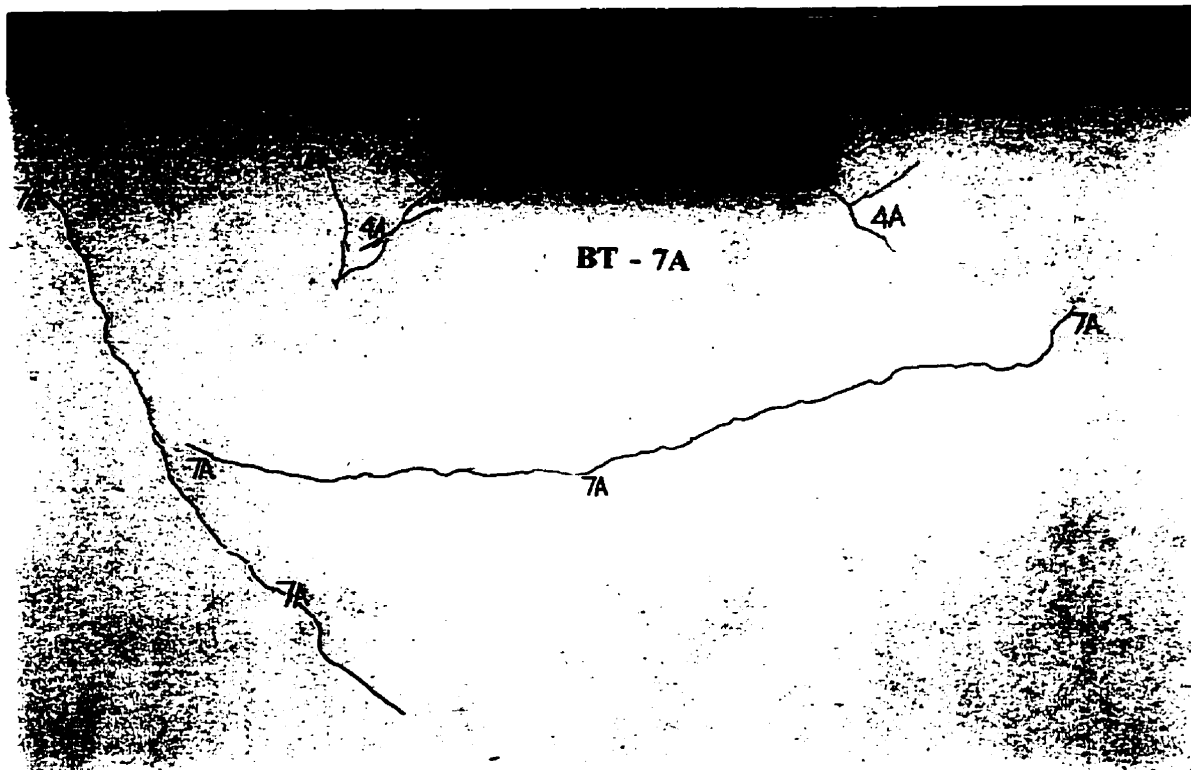
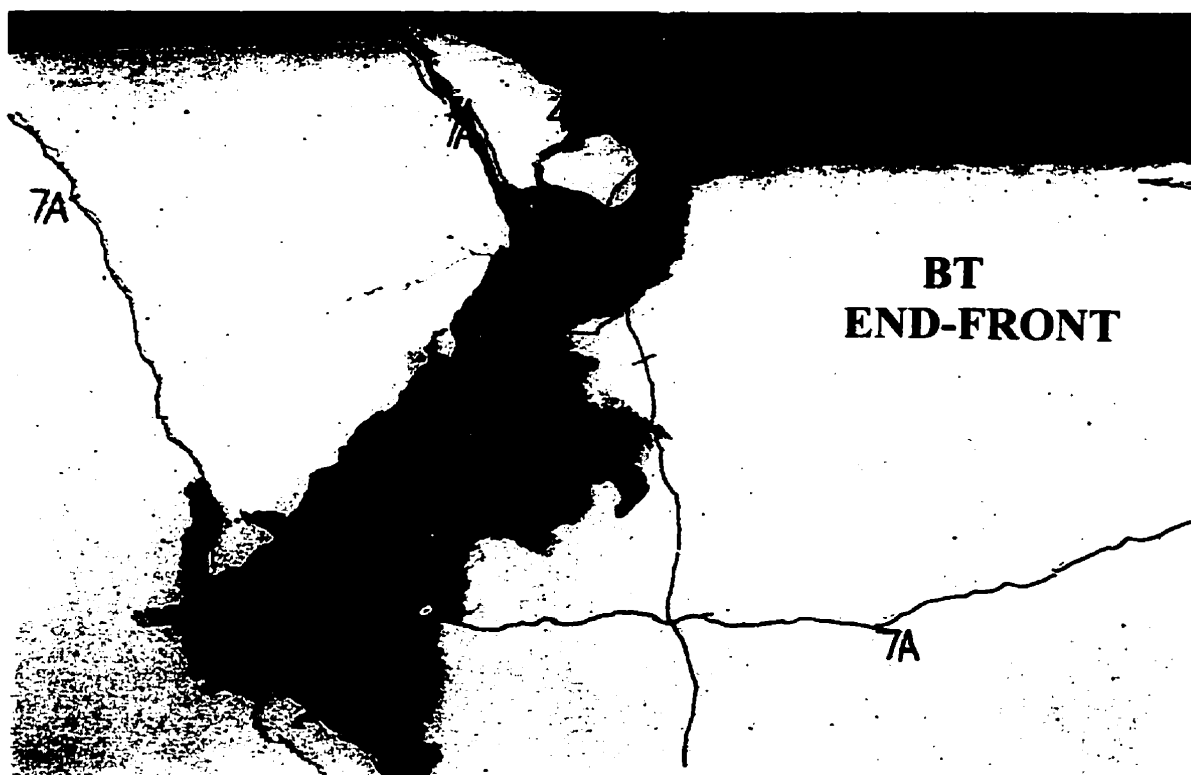


Figure 3.46 Strain distribution along the length of bar prior to failure of Specimen BT



(a) Connection at first yield, Δ_y , $P=140$ kN



(b) Connection after failure

Figure 3.47 Photographs of Specimen BT

3.4.4 Specimen BST1

The loading history (T/V ratio = 0.7) and the associated displacements are given in Table 3.10. Figures 3.48 and 3.49 show the hysteresis curves and the loading histories for the shear and the axial response, respectively. The measured strains along the length of the reinforcing bar for the shear and the axial loads are presented in Figures 3.50 and 3.51, respectively. Figures 3.52(a) and 3.52(b) show the photographs of the specimen at the yielding stage and after failure, respectively. The distribution of strains with the distance along the length of anchor bar at the ultimate load is shown in Figure 3.53. The following notes are made based on the observations during the test and from the analysis of experimental results.

Cycles 1-3, $V=45$ kN $T=30$ kN No cracks was noted. The connection response was linear elastic resulting in no energy dissipation.

Cycles 4-6, $V=75$ kN $T=50$ kN The first splitting crack appeared along the tension leg, however it did not change the linearity of load-deflection curves. The displacements were small

Cycles 7-9, $V=90$ kN $T=65$ kN The reinforcing bar yielded (Figures 3.50 and 3.51). The displacements at the yield stage (Δ_y) were 1.2 mm and 1.5 mm for horizontal and axial loads, respectively. Due to yielding, significant cracks occurred in the tension zone and one splitting crack appeared along the compression leg [Figure 3.52(a)], and the stiffness of connection decreased (Figures 3.48 and 3.49). The maximum splitting and diagonal crack width were 0.33 mm and 1.0 mm , respectively. The area under the loops increased during the positive load cycle where in negative direction, the area was very small. However, the dissipated energy was noticeable due to development of yielding of the connection anchor bar.

Cycle 10, $V=140$ kN $T=70$ kN The ultimate strength of 140 kN and 70 kN for combined shear and axial force was achieved. Then the shear and axial loads dropped to 65 kN and 20 kN, respectively. The specimen was then subjected to loads of -140 kN and -100 kN for shear and axial loads, respectively. There was an attempt to load the specimen for another cycle when the connection could not resist more load and failed. The failure mode was due to splitting and spalling of the concrete along the tensile leg,

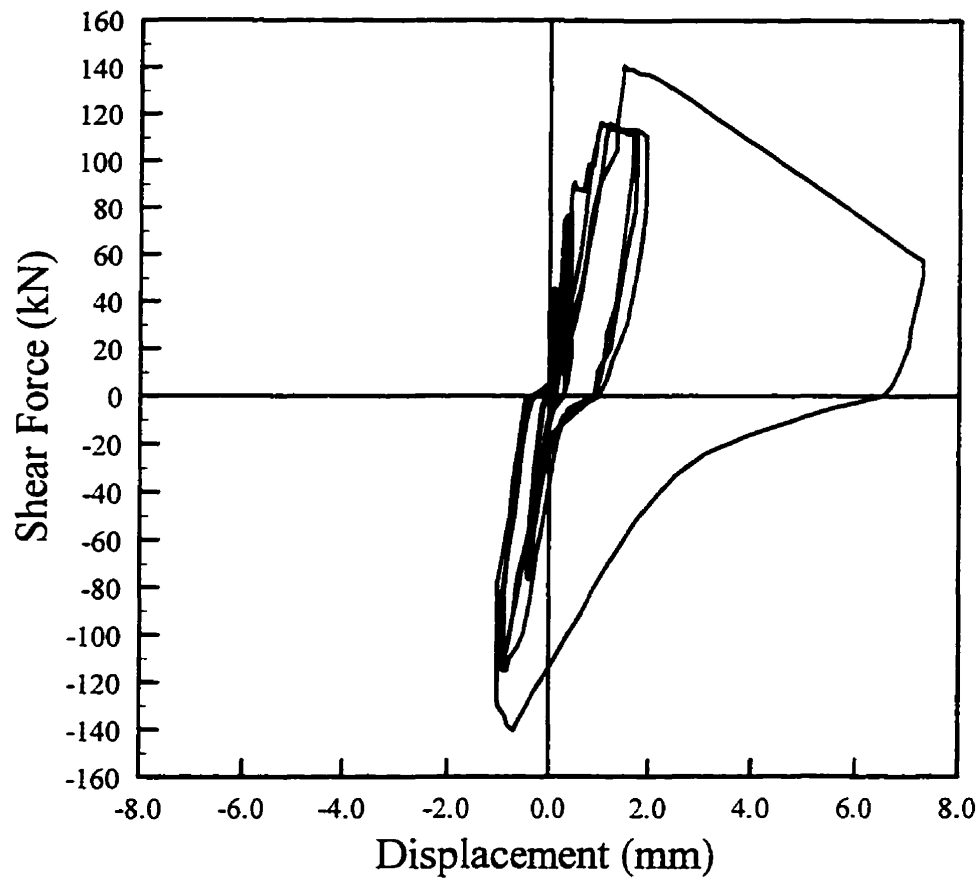
breaking away of the concrete around the steel angle and crushing of the concrete in front of the 90° hook [Figure 3.49(b)]. The ultimate displacements were 7.3 mm and 6.63 mm (for horizontal and axial displacements), respectively.

The distribution of strain along the anchor bar legs at the ultimate load show that about half of the reinforcing bar in the tension leg had yielded (Figure 3.53). The results show that under combined cyclic axial and shear loads ($T/V=0.7$), the strength of connection decreased to about 70 % and 47% of those for pure cyclic shear and pure cyclic axial strengths, respectively. The hysteresis response shows larger loops in the positive direction (combination of shear and tensile axial load) which was due to axial load and contribution of standard hooks resulting in developing of yielding along the reinforcing bar. The smaller loops in the negative direction (shear and compression axial loads) were due to the compressive axial load , imperfect closing of the cracks and the relatively small deformability of concrete subjected to compressive stresses. Based on the ultimate horizontal and axial displacements, a displacement ductility ratio of 5.0 was attained.

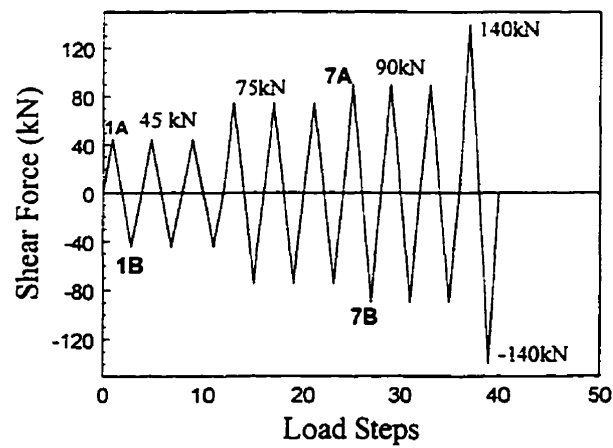
Table 3.20 Summary of loading history for Specimen BST1

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.3 , -0.3	±45	+0.3 , +0.02	±30
Cracked	4A - 6B	+0.4 , -0.5	±75	+0.5 , +0.04	±50
±1 Δ_y	7A - 9B	+1.9 , -1.0	±90	+1.9 , +0.1	±65
5 Δ_y , Failure	10A - 10B	+7.3	±140	+6.6	+70 , -100

* Maximum displacement at the peak load of the last cycle



(a) Hysteresis response



(b) Loading History

Figure 3.48 Hysteresis shear response and loading history for Specimen BST1

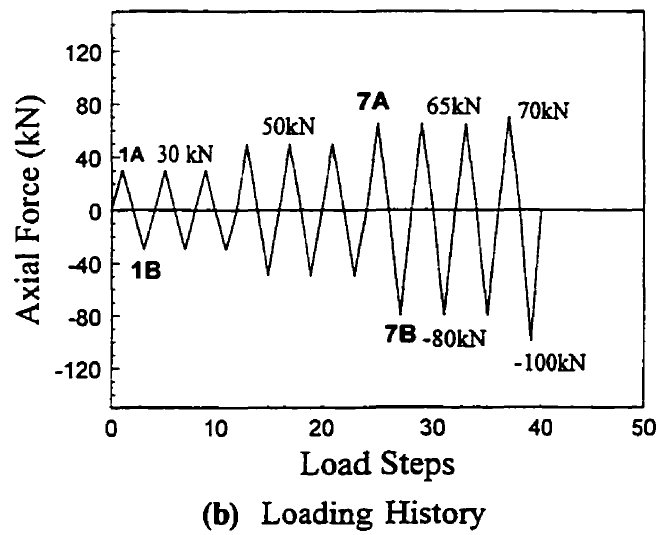
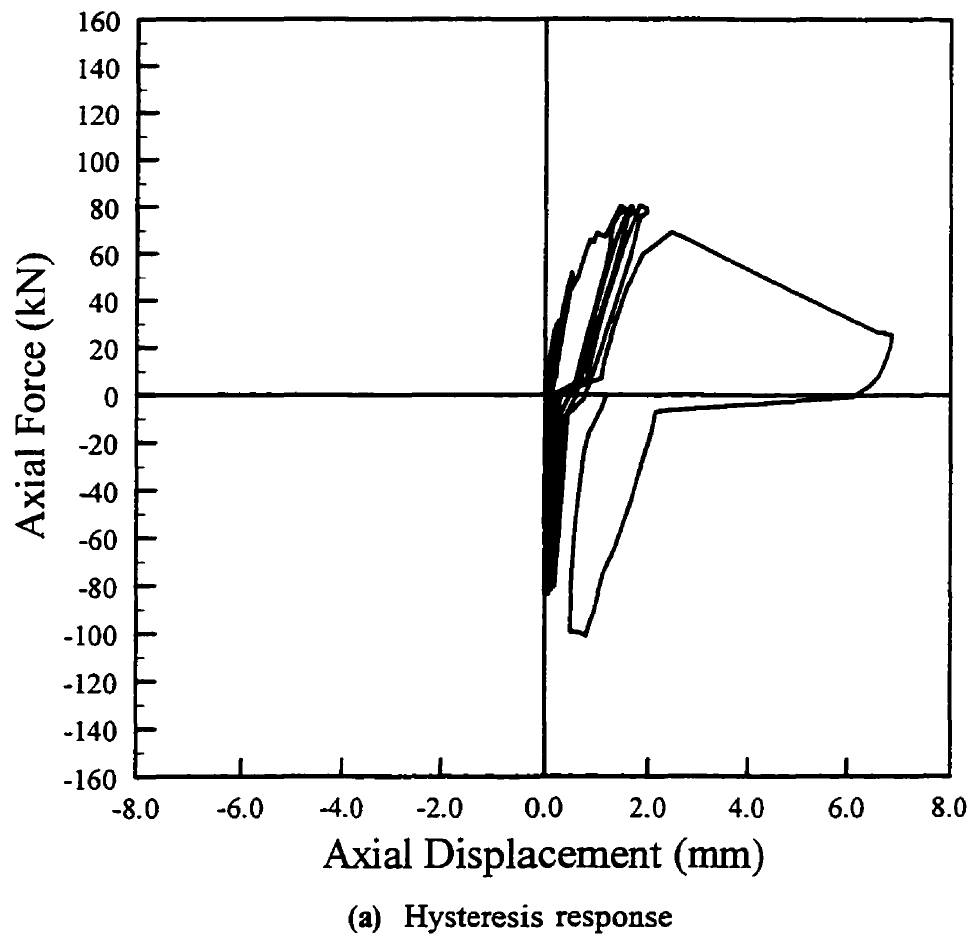
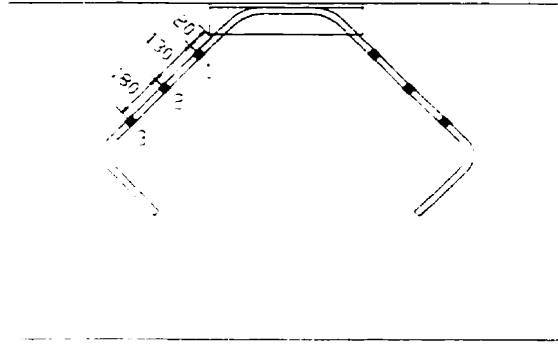
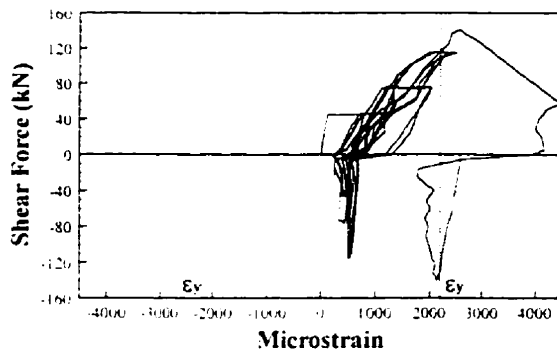


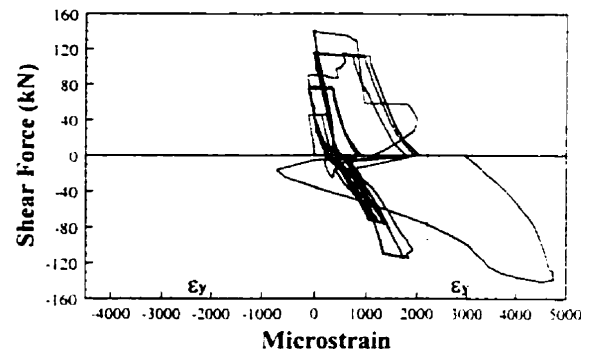
Figure 3.49 Hysteresis axial response and loading history for Specimen BST1



(a) Location of strain gauges

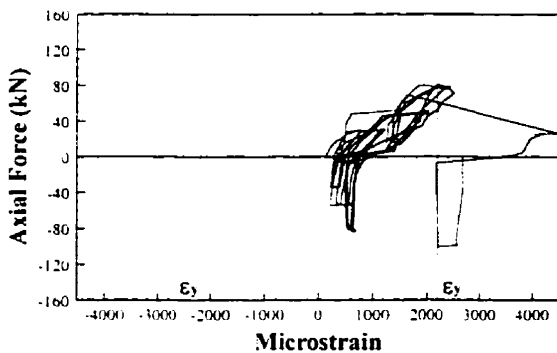


(b) Strains at Location 1. Shear response

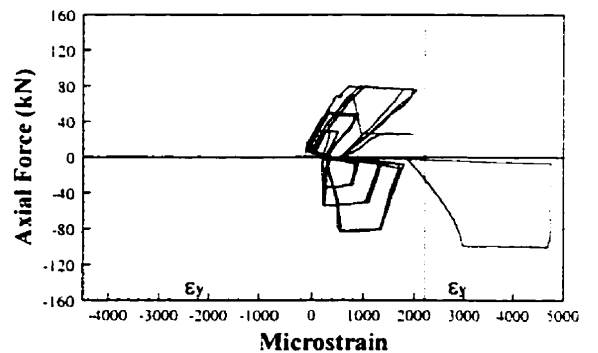


(c) Strains at Location 4. Shear response

Figure 3.50 Strains along length of anchor in Specimen BST1 (Shear response)



(b) Strains at Location 1. Axial response

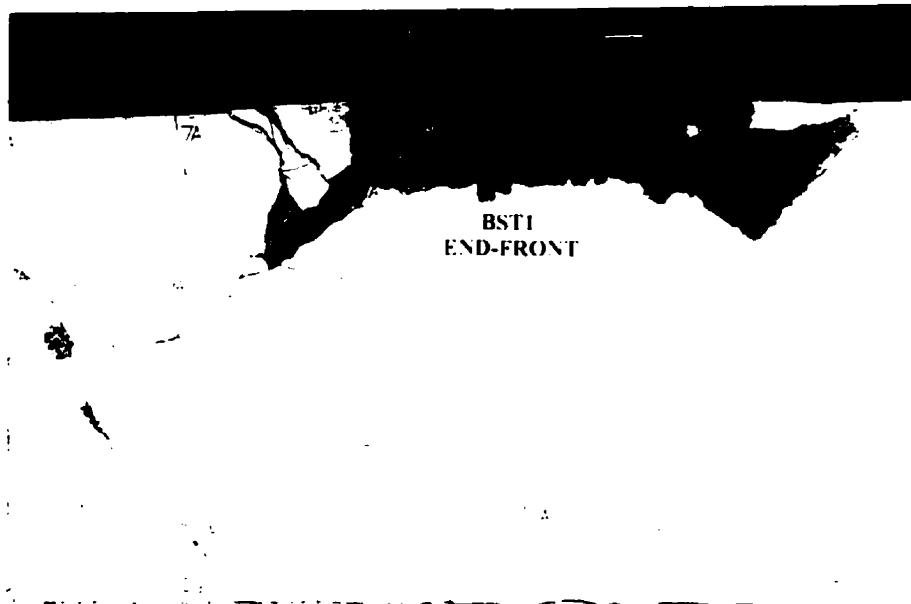


(c) Strains at Location 4. Axial response

Figure 3.51 Strains along length of anchor in Specimen BST1 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}}=90$ kN, $P_{\text{axial}}=65$ kN



(b) Connection after failure

Figure 3.52 Photographs of Specimen BST1

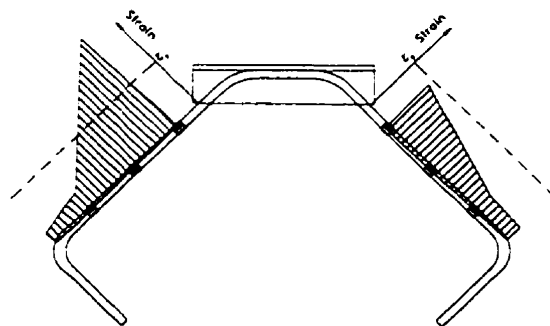


Figure 3.53 Strain distribution along the length of bar prior to failure of Specimen BST1

3.4.5 Specimen BST2

The loading history (T/V ratio = 1.0) and the associated displacements are presented in Table 3.11. The load-deflection response and the loading histories for the shear and the axial loads are shown in Figures 3.54 and 3.55, respectively. The strains along the length of the steel anchor bar for shear and axial load are shown in Figures 3.56 and 3.57, respectively. The photographs of the specimen at the yielding level and after failure are shown in Figures 3.58(a) and 3.58(b), respectively. The profile of the strains with the distance along the length of anchor bar at the ultimate load is shown in Figure 3.59. The following comments are made, based on the observation during the experiment and from analysis of the experimental results.

Cycles 1-3, $V=T=40$ kN The load-displacement curves remained linear. No cracks were observed.

Cycles 04-06, $V=T=60$ kN Small splitting cracks appeared along both tension and compression legs plus one diagonal crack around the 90° hook in the tension zone. Due to cracking, the slope of load-deflection curves changed, especially for axial load response (Figures 3.54 and 3.55). The area under loops was small showing little energy dissipation.

Cycles 7-9, $V=T=85$ kN The reinforcing anchor yielded at Location 1 (Figures 3.56 and 3.57), where the measured displacement (Δ_y) for horizontal and axial were 0.95 mm and 1.6 mm, respectively. The measured crack widths were 0.8 mm for both splitting and diagonal cracks. The photograph of the connection at the yield stage is shown in Figure 3.58(a). During the positive loading (shear and tensile axial loading), the hysteresis loops were large while for the negative loads (shear and compression loadings) the loops were very narrow. However, significant energy was dissipated due to yielding and crack propagation (Figures 3.54 and 3.55).

Cycles 10-12, $V=T=100$ kN This stage was implemented using displacement control with a maximum deflection equal to $2\Delta_y$. The horizontal and axial displacements were 2.4 mm and 3.1 mm, respectively. The steel anchor yielded at Location 2. During the following cycles of this load stage, the displacement were close to $2\Delta_y$, resulting in a decrease in the axial load (Figure 3.55). This indicates a decrease in stiffness and strength due to the cyclic loading. During Cycle 12, when the shear force was applied in the

negative direction, the steel anchor yielded at Location 4 [Figure 3.56(c)]. The area under the hysteresis loops increased, especially in the positive direction, showing the capability of the connection to dissipate energy through development of yielding along the reinforcing bar.

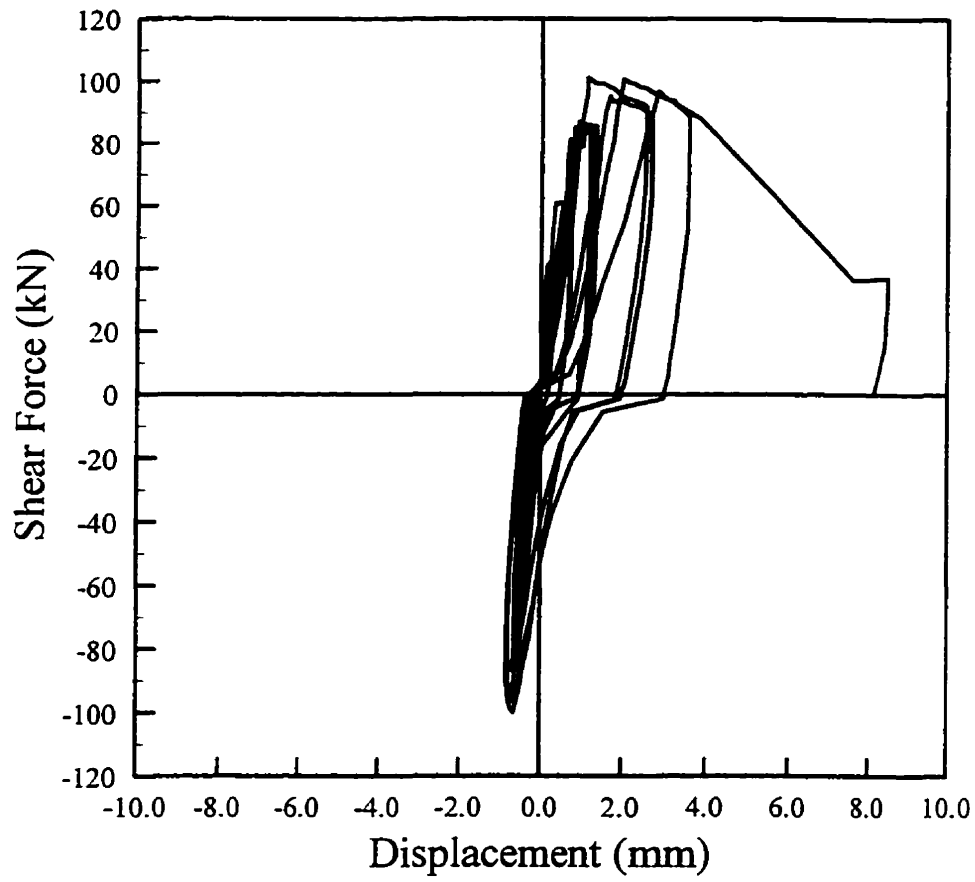
Cycles 13 $V=99$ $T=55$ kN The specimen failed at this stage. The mode of failure was due to splitting of the concrete along the straight part of tensile leg, crushing of the concrete in front of the 90° hook in the tension zone, and crushing of the concrete around the steel angle [Figure 3.58(b)]. The failure mode was close to that for the Specimen BST1. The ultimate horizontal and axial displacement were measured to be 6.9 mm and 7.1 mm, respectively, representing a displacement ductility ratio of 5.

The distribution of strains along the anchor leg at the ultimate load shows that about two-thirds of the reinforcing bar in the tension leg had yielded (Figure 3.59). The maximum strength of 100 kN for both shear and axial loading was achieved. Based on the experimental results, under combined cyclic axial and shear ($T/V=1.0$), the strength of the connection reduced to about 50% and 67% of those for pure cyclic shear and pure cyclic axial strengths, respectively. The hysteresis response showed large loops, especially in the positive direction, which was mostly due to the contribution of the standard hooks, resulting in the development of yielding in the steel anchor bar. A displacement ductility ratio of 5 was achieved.

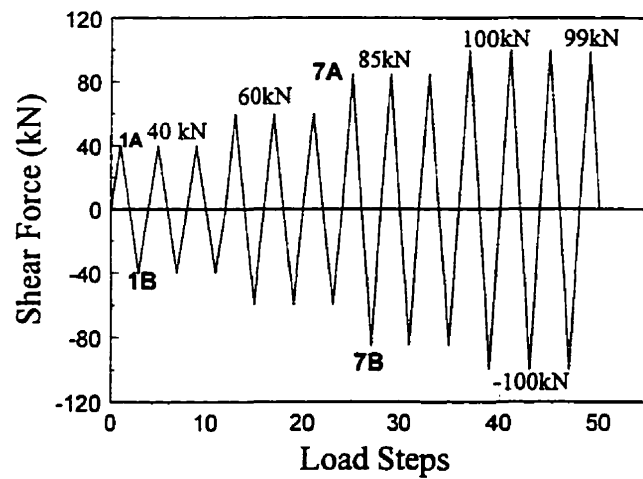
Table 3.11 Summary of loading history for Specimen BST2

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.3 , -0.3	±40	+0.4 , +0.1	±40
Cracked	4A - 6B	+0.7 , -0.4	±60	+1.0 , +0.1	±60
±1 Δ_y	7A - 9B	+1.4 , -0.7	±85	+1.8 , +0.2	±85
±2 Δ_y	10A - 12B	+3.6 , -0.8	±100	+3.4 , +0.4	±100
Failure, 5 Δ_y	13A	+6.9	+99	+7.1	+55

* Maximum displacement at the peak load of the last cycle

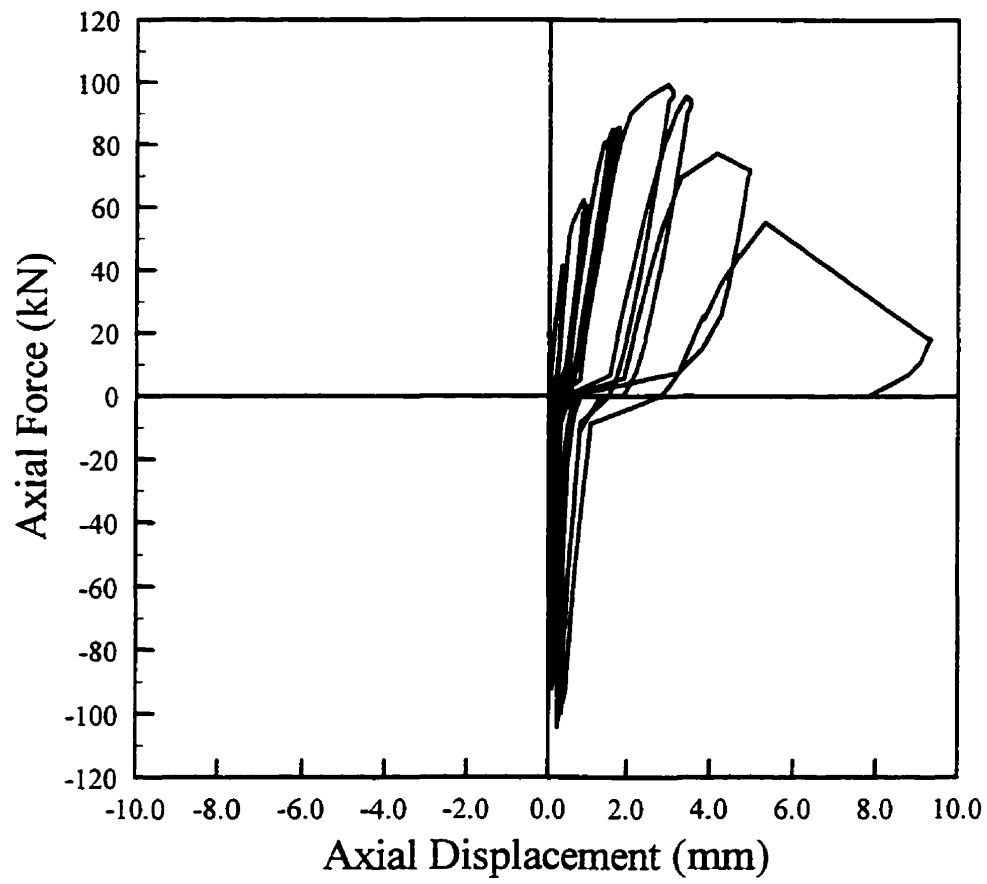


(a) Hysteresis response

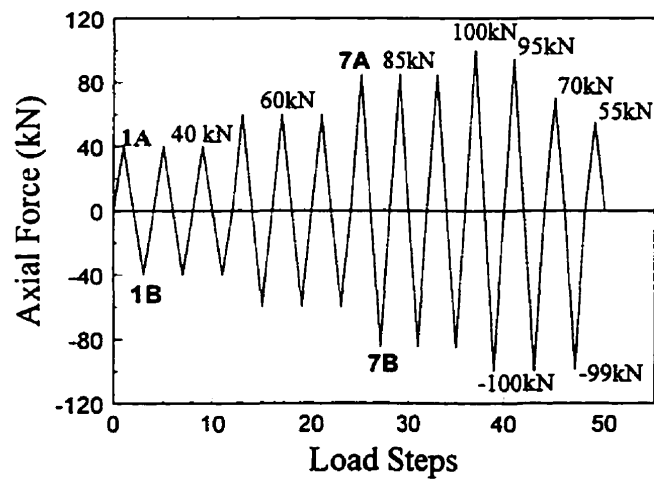


(b) Loading History

Figure 3.54 Hysteresis shear response and loading history for Specimen BST2

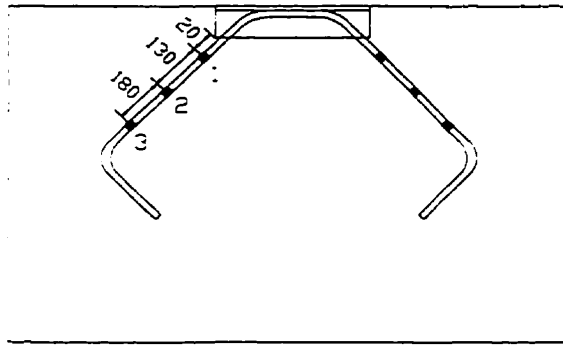


(a) Hysteresis response

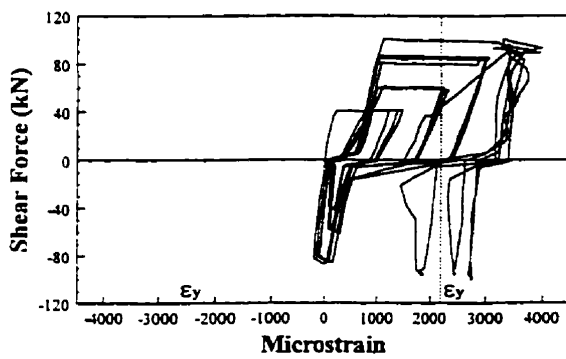


(b) Loading History

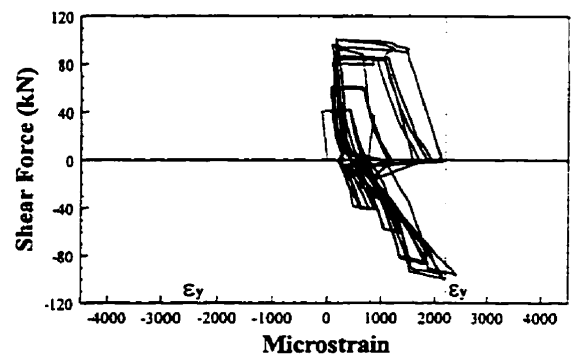
Figure 3.55 Hysteresis axial response and loading history for Specimen BST2



(a) Location of strain gauges

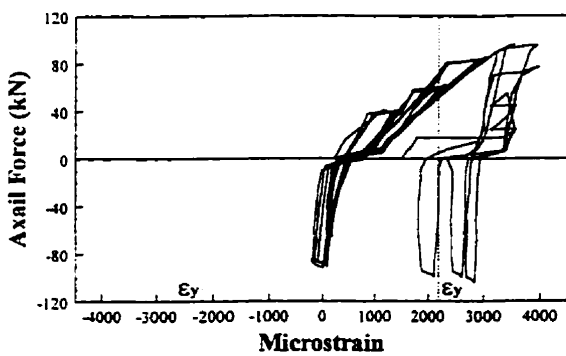


(b) Strains at Location 1, Shear response

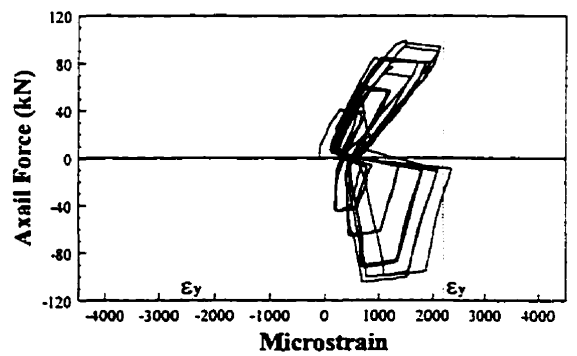


(c) Strains at Location 4, Shear response

Figure 3.56 Strains along length of anchor in Specimen BST2 (Shear response)



(b) Strains at Location 1, Axial response



(c) Strains at Location 4, Axial response

Figure 3.57 Strains along length of anchor in Specimen BST2 (Axial response)

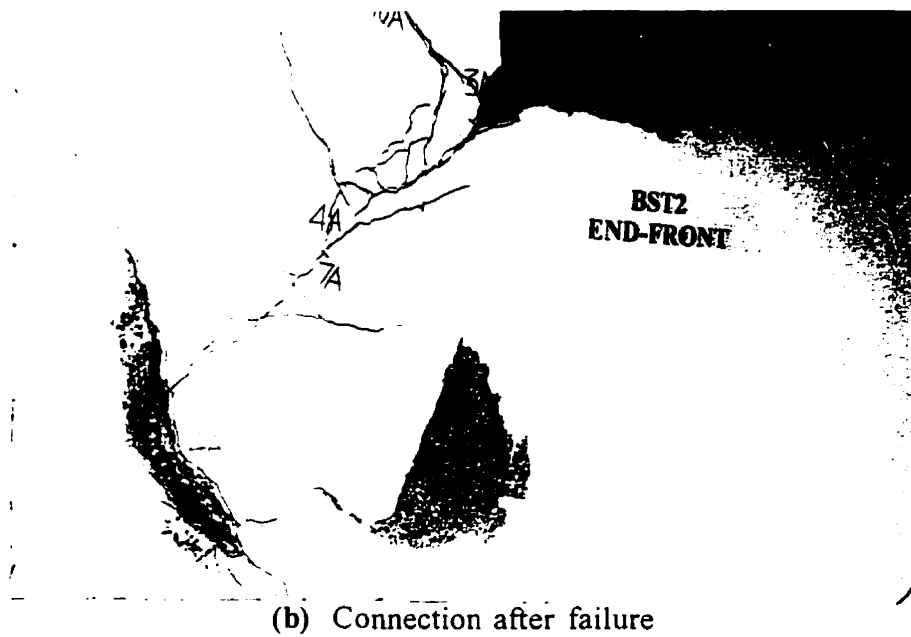
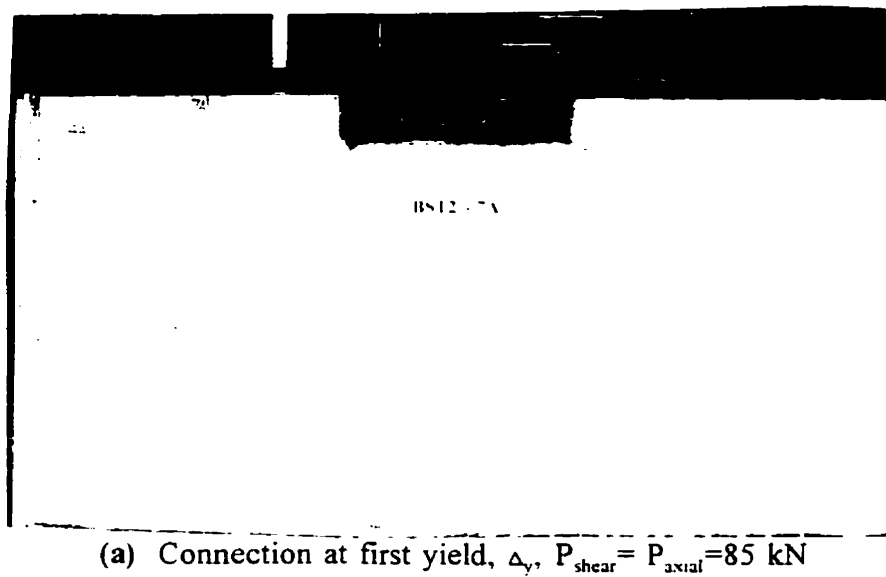


Figure 3.58 Photographs of Specimen BST2

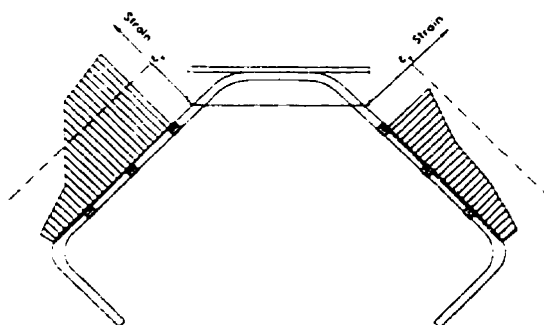


Figure 3.59 Strain distribution along the length of bar prior to failure of Specimen BST2

3.4.6 Specimen BST3

Table 3.12 summarizes the loading history (T/V ratio = 0.3) and the associated displacements. Figures 3.60 and 3.61 show the hysteresis response and the loading histories for the shear and the axial loading. The measured strains in the steel anchor bar for shear and axial forces are presented in Figures 3.62 and 3.63, respectively. Figures 3.64(a) and 3.64(b) show the photographs of the test specimen at the yielding stage and after failure, respectively. Figure 3.65 shows the distribution of the strain along the length of the steel bar at the ultimate load. The following observation and results were noted during testing and analysis of the experimental results:

Cycles 1-3, $V=60$ kN $T=20$ kN The response was basically linear elastic without any cracking.

Cycles 4-6, $V=90$ kN $T=30$ kN Small splitting crack appeared along the tension leg, mostly due to shear force, however, the slope of load-deflection curves did not change significantly (Figures 3.60 and 3.61). The area under the hysteresis loops and the displacements were very small indicating almost no energy dissipation.

Cycles 7-9, $V=125$ kN $T=35$ kN The first yielding occurred at Location 1 along the reinforcing bar [Figures 3.62(a) and 3.63(a)]. The measured horizontal and axial displacements (Δ_y) were 1.2 mm and 1.1 mm, respectively. The anchor bar yielded at Location 4 during the negative loading ([Figures 3.62(b) and 3.63(b)]. Additional diagonal cracks formed in the tension zone [photograph in Figure 3.64(a)]. The maximum crack width were 0.3 mm and 0.4 mm for the splitting and the diagonal, respectively. The area under the hysteresis loops were small resulting in insignificant energy being dissipated.

Cycles 10-12, $V=150$ kN $T=45$ kN This stage was implemented using displacement control with maximum displacement equal to $2\Delta_y$. The measured horizontal and axial displacements were 2.3 mm and 2.2 mm, respectively. During the last cycle of this load stage the reinforcing bar yielded at Location 2. The area under the loops increased significantly due to the development of yielding along the bar (general yielding), which indicates the energy dissipation capability of the connection. The maximum crack widths were measured to be 0.6 mm and 1.0 mm for the splitting and diagonal cracks, respectively.

Cycles 13-14, $V=140$ kN $T=10$ kN This stage involved using displacement control with a maximum displacement equal to $3.5\Delta_y$. The displacements were 4.3 mm and 3.9 mm for horizontal and axial loads, respectively. During the last cycle as the shear force of 130 kN was applied, the specimen failed due to splitting and spalling of concrete along the straight part of the tensile leg along with the crushing of the concrete in front of the 90° hook [Figure 3.64(b)]. The failure mode was similar to that for the Specimen BS. The ultimate horizontal and axial displacement were measured to be 8.1 mm and 4.1 mm, respectively.

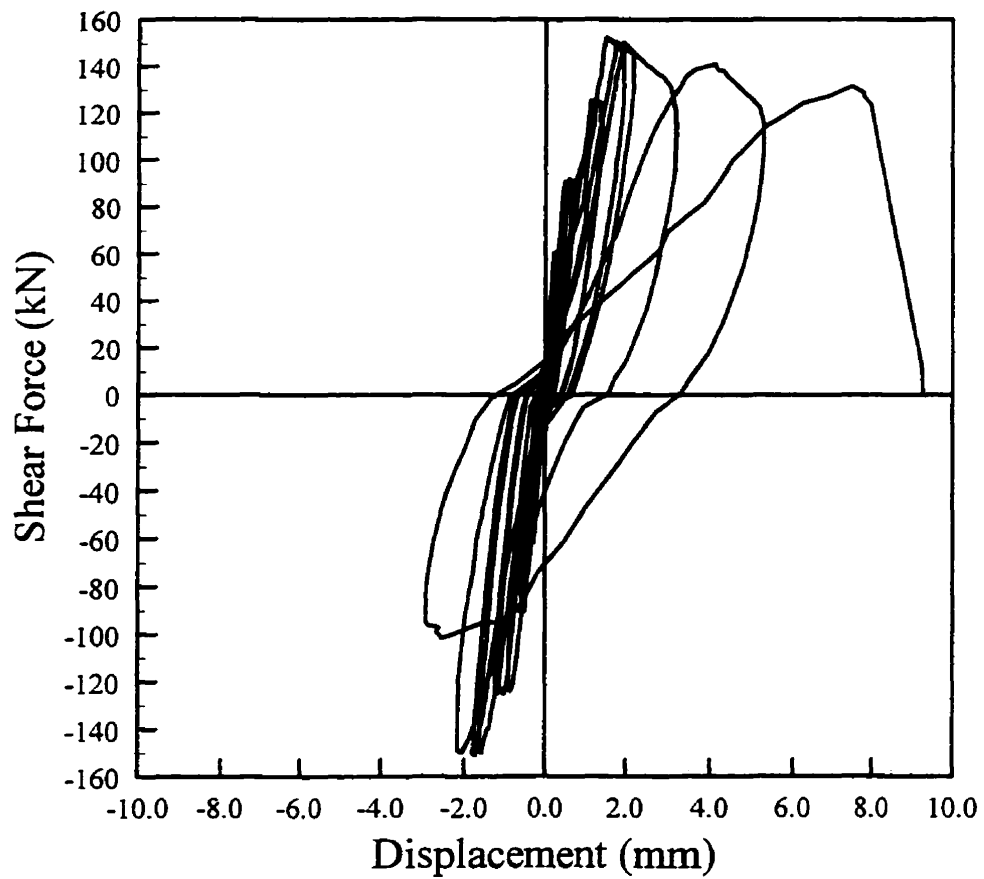
The profile of strains along the anchor leg at the ultimate load shows that more than two-thirds of the reinforcing bar in the tension leg had yielded (Figure 3.65). The maximum shear and axial strengths of 150 kN and 45 kN, respectively, were achieved.

The hysteresis response shows large loops after general yielding of the reinforcing anchor, which indicates the capability of energy dissipation. Based on the ultimate displacements, a displacement ductility ratio of 5.6 was attained. The experimental results show that when 30% of the shear force is applied in the axial direction in a cyclic manner, the strength of connection reduces to about 75% of that for pure cyclic shear strength.

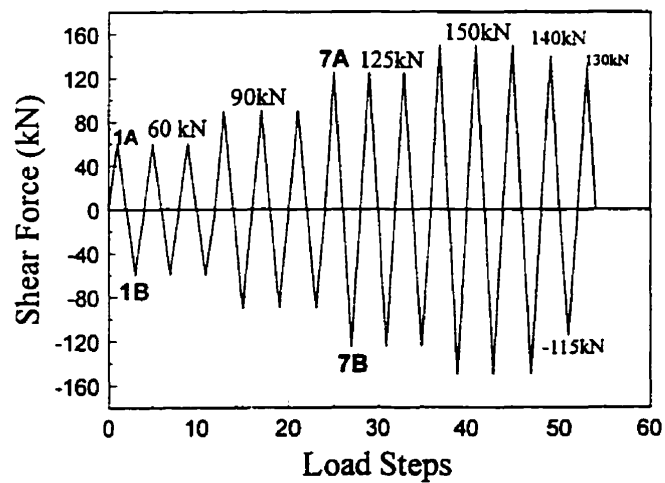
Table 3.12 Summary of loading history for Specimen BST3

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.4 , -0.3	±60	+0.5 , +0.5	±20
Cracked	4A - 6B	+0.7 , -0.6	±90	+1.0 , +0.6	±30
±1 Δ_y	7A - 9B	+1.4 , -1.3	±125	+1.5 , +1.0	±35
±2 Δ_y	10A - 12B	+2.7 , -2.2	±150	+3.4 , +1.1	±45
±3.5 Δ_y	13A - 13B	+4.3 , -3.9	+140 , -115	+3.9 , +1.5	+10
5.6 Δ_y , Failure	14A	+8.1	+130	+4.1	-

* Maximum displacement at the peak load of the last cycle

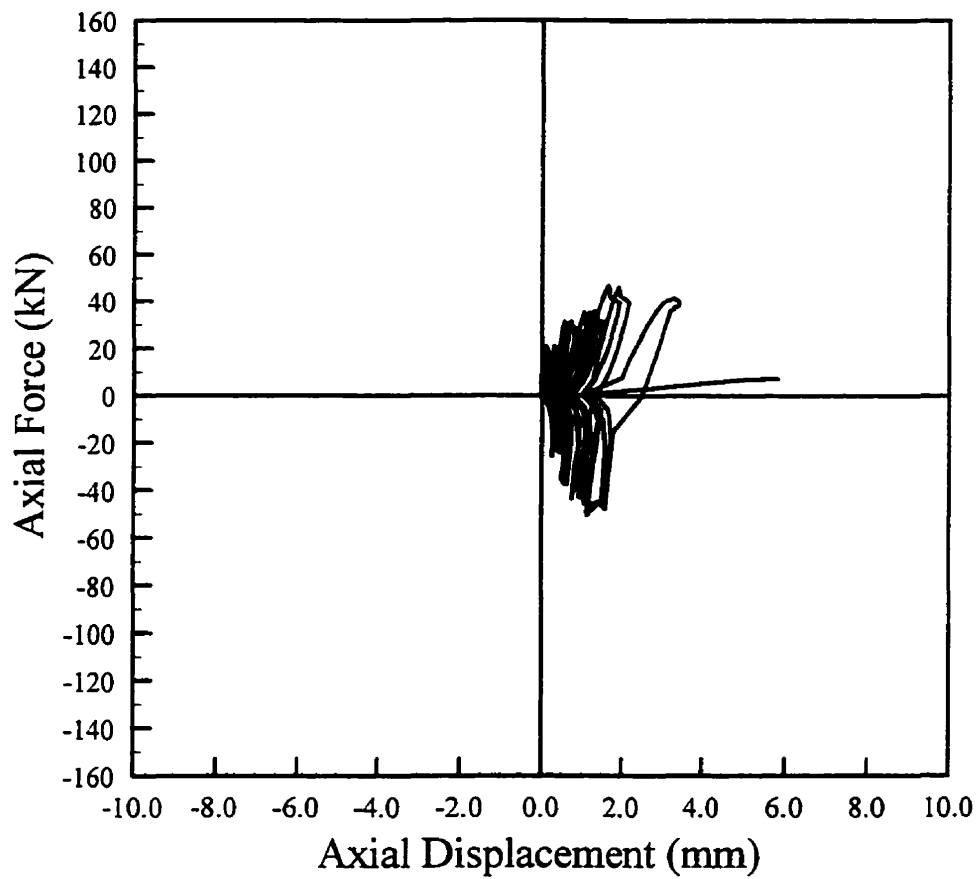


(a) Hysteresis response

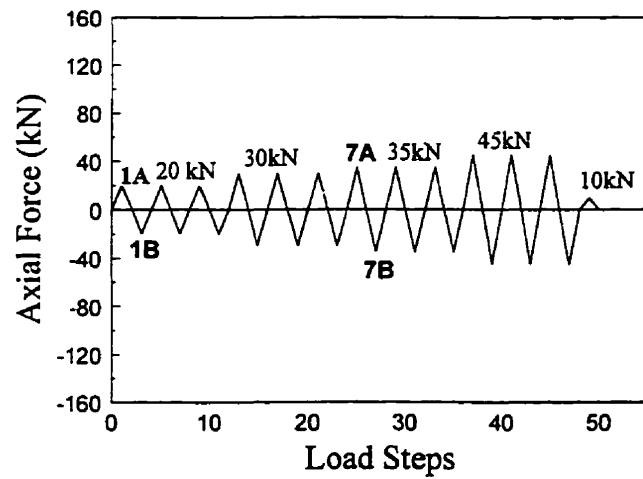


(b) Loading History

Figure 3.60 Hysteresis shear response and loading history for Specimen BST3

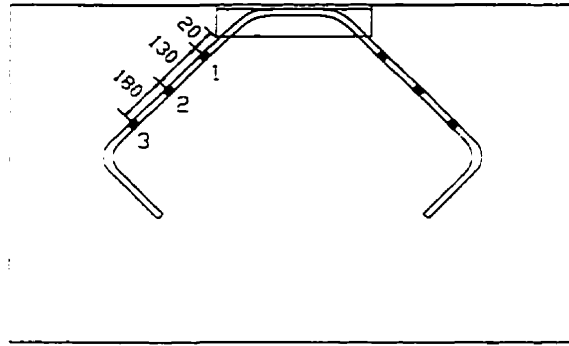


(a) Hysteresis response

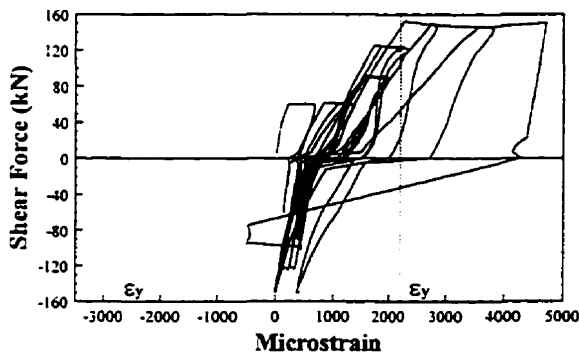


(b) Loading History

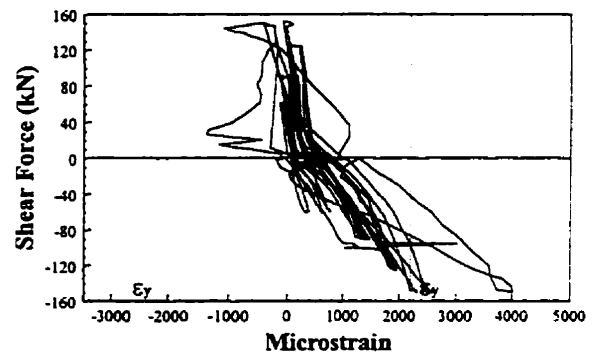
Figure 3.61 Hysteresis axial response and loading history for Specimen BST3



(a) Location of strain gauges

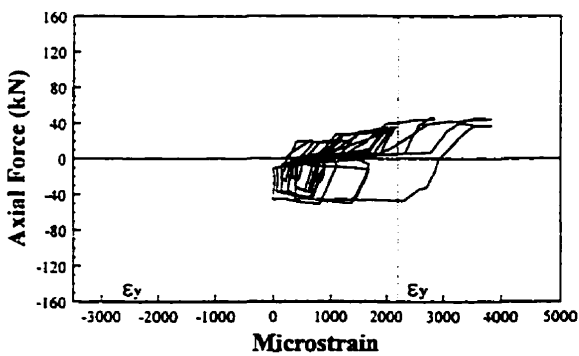


(b) Strains at Location 1, Shear response

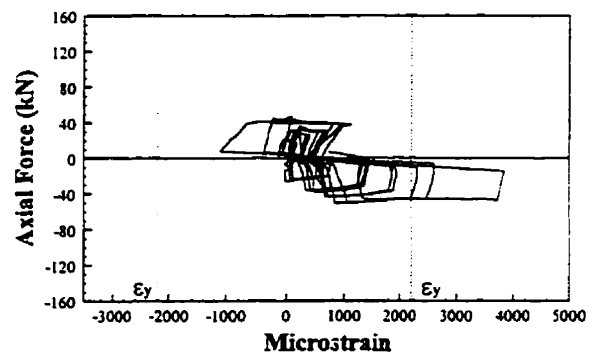


(c) Strains at Location 4, Shear response

Figure 3.62 Strains along length of anchor in Specimen BST3 (Shear response)

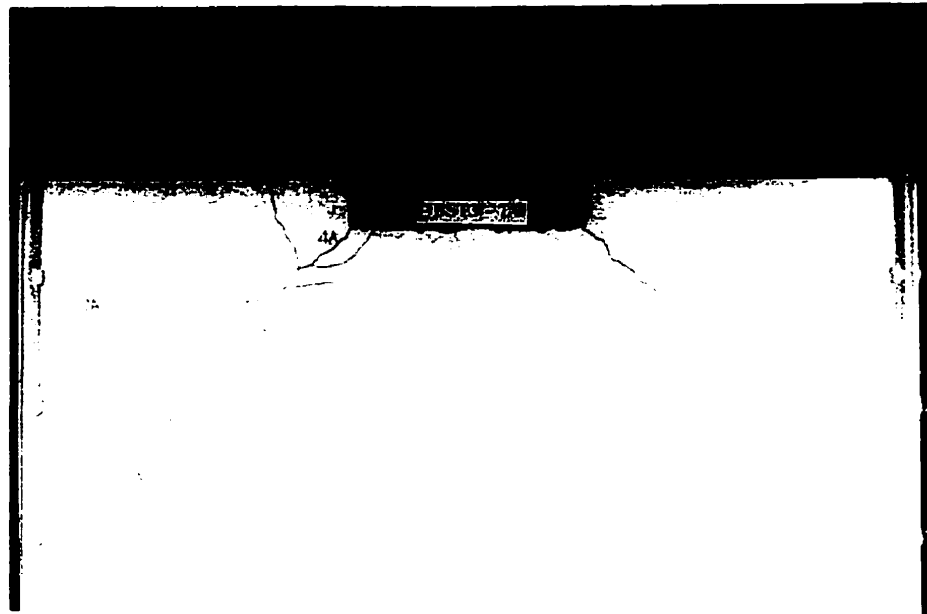


(b) Strains at Location 1, Axial response

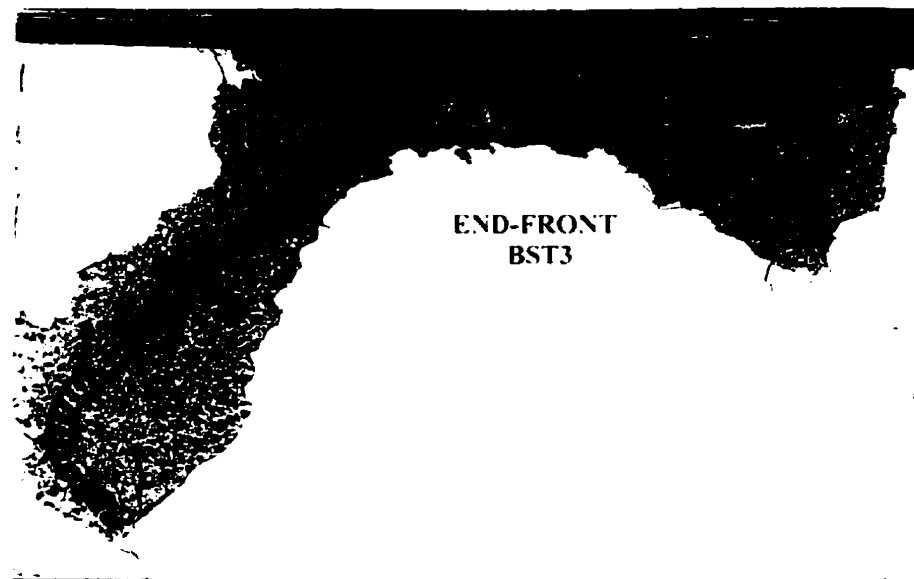


(c) Strains at Location 4, Axial response

Figure 3.63 Strains along length of anchor in Specimen BST3 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}} = 152 \text{ kN}$, $P_{\text{axial}} = 45 \text{ kN}$



(b) Connection after failure

Figure 3.64 Photographs of Specimen BST3

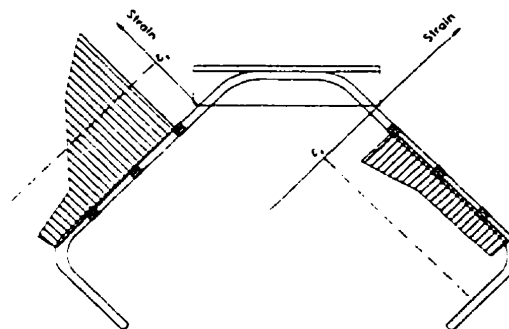


Figure 3.65 Strain distribution along the length of bar prior to failure of Specimen BST3

3.4.7 Interaction Diagram for Series B

For safe design of precast floor systems, it is necessary to know the load carrying capacity of the connections under the various combinations of shear and axial forces induced during a seismic activity. This detailed investigation was undertaken to determine the interaction diagram and the associated equation between the shear and the axial loads acting simultaneously on the connection.

The general behaviour and the ultimate capacities of the specimens in Series B tested under the different combination of cyclic shear and axial loadings were discussed in previous sections (Sections 3.3.2 to 3.3.6). From the experimental results as summarized in Table 3.13, the maximum cyclic shear strengths, expressed as V/V_0 are plotted versus the axial cyclic ratios, T/T_0 , in Figure 3.66. The experimental interaction equation has been fitted to the experimental result as:

$$\left(\frac{V}{V_0}\right) + \left(\frac{T}{T_0}\right)^{1.5} \leq 1.0 \quad (2)$$

where V is the applied shear force, V_0 is the maximum force that the connection can resist in pure cyclic shear, T is the applied axial force, and T_0 is the maximum force that the connection can resistance in pure axial loads. The material resistance factors must be included for the equation to be applicable for design (see Chapter 7).

It can be seen from Figure 3.66 that the cyclic shearing strength decreases when an cyclic axial force is applied, basically because the axial forces cause tensile stresses in the concrete along the reinforcing bar (bond stress), and, also, stresses the concrete surrounding the steel angle, weakening the bond strength when the shear forces are applied. Besides, the cracking load and shear stiffness reduce significantly due to the application of the axial forces. Similarly, the cyclic axial strength decreases when an cyclic shear force is applied.

Table 3.13 Summary of interaction test results for Series B

Test type	Ratio of axial to shear (T/V)	Ultimate strength (kN)		Ratios	
		Shear (kN)	Axial (kN)	V/V	T/To
BS	-	200	-	1.00	0.0
BST1	0.7	140	80	0.705	0.530
BST2	1.0	100	95	0.505	0.629
BST3	0.3	150	41	0.750	0.272
BT	-	-	150	0.0	1.0

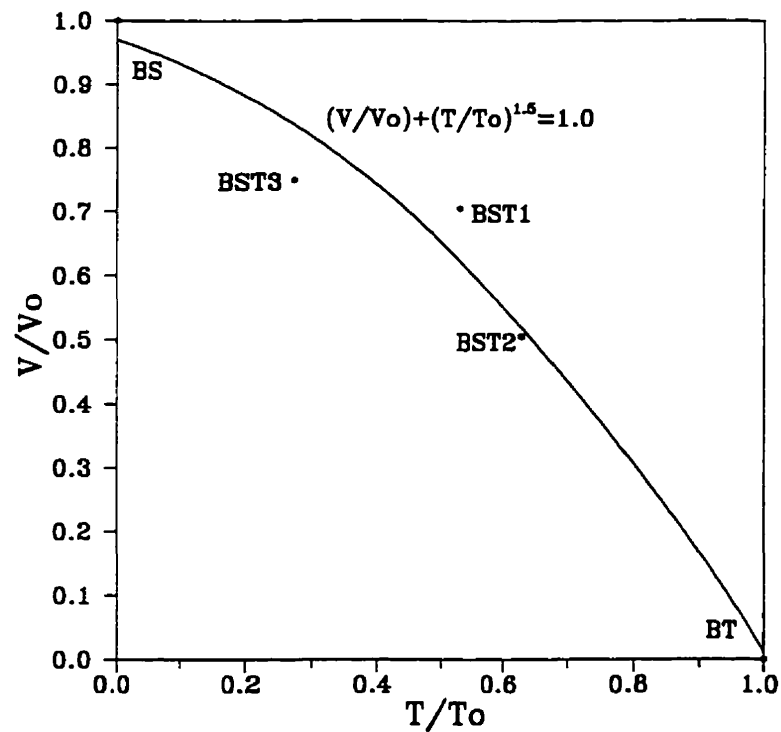


Figure 3.66 Normalized Interaction Diagram for Series B

Chapter 4

Experimental Results -Series C and D

4.1 Introduction

The complete response of two sets of six identical connections (Series C and D), tested to destruction under reversed cyclic loads are described in this chapter. The hysteresis response of these connections under reversed cyclic shear and axial loading (tension and compression), monotonically increasing load responses, shear force-axial force interaction curves, specimens displacements, and measured strains at six or nine locations on the reinforcing bar anchor are reported.

The hysteresis responses of the connections presented in this chapter require some explanations. In order to investigate the cyclic response of the connections to combined axial and shear forces, three specimens of each series were subjected to cyclic combined shear and axial forces with three different values of T/V ratio. The axial forces were applied with a quarter cycle delay after the application of the shear forces.

The following figures are presented for each connection specimen:

1. Monotonically applied loads vs. horizontal displacement(monotonical shear response).
2. Cyclic applied load vs. horizontal displacement response (cyclic shear response).
3. Cyclic applied load vs. vertical displacement response (cyclic axial response).
4. Applied load vs. measured strain in the reinforcing bar anchors.
5. Measured strain vs. distance along the embedded anchors at ultimate load.
6. Photographs of the specimen at different load stages.
7. Shear force-axial force interaction diagrams for each series.

4.2 Test Series C: Connection with straight deformed steel bar welded to an angle

This connection was reinforced with two No 15 deformed bar oriented at 90° , welded to the flange of the 10 mm thick steel angle (see Figure 2.17). A comprehensive description of the connection is presented in Section 2.4. The design and determination of ultimate shear capacity was performed based on the shear-friction mechanism, while for the connection subjected to axial loads, the strength of the connection was determined based on the bond strength at the steel bar - concrete interface. The connection was designed with consideration for the bar development length required for cyclic loads (ACI, 1995). Six identical specimens were constructed and tested subjected to different load combination (T/V ratios), one under monotonically increasing shear (Specimen CMS) and the remainder (Specimens CS, CT, CST1, CST2 and CST3) subjected to combined cyclic shear and axial loads for the various values of the T/V ratio.

4.2.1 Specimen CMS

Figure 4.1 shows the shear force-horizontal displacement curve for the Specimen CMS. The load versus measured strains along the length of bar are presented in Figure 4.2. The distribution of the strains prior to failure versus distance along the length of bar is shown in Figure 4.3. Figures 4.4(a) and 4.4(b) show photographs of connection at the yield stage and after failure, respectively. The following important behavioural aspects were noted during testing, and analysis of the experimental results.

Stage 1 70 kN The first crack appeared along the left steel bar due to dowel action. Before the appearance of the crack, the load-deflection remained linear (Figure 4.1). The horizontal displacement and the crack width were 0.35 mm and 0.15 mm, respectively.

Stage 2 100 kN A splitting crack developed along the other (right) anchor bar, and a diagonal crack formed at the ends of the bars. This resulted in sudden increase in displacement to about 1.2 mm (see Figure 4.1).

Stage 3 125 kN The reinforcing bar yielded at Location 1 (Figure 4.2). Two more diagonal cracks appeared across the concrete panel, one in the compression zone (right side) and the other in the tension zone (left side) [Figure 4.4(a)]. The measured displacement at the yield load and the maximum crack width were $\Delta_y=2.4$ mm and 1.0

mm, respectively. When the load was increased to 144 kN, the reinforcing bar yielded at Location 2 (Figure 4.2). The connection underwent a significantly increased displacement. The maximum displacement was about $2\Delta_y=4.8$ mm (Figure 4.1). Horizontal cracks were observed between the two reinforcing bars, showing that the concrete between the bars was subjected to flexure, resulting in cracking. The maximum crack width was 1.25 mm along the length of the reinforcing bar.

Stage 4 152 kN As the load was increased further, the slope of the shear force-displacement curve kept on decreasing until the ultimate load of 152 kN (Figure 4.1). The load dropped a little and then the connection failed at a load of 135 kN. The ultimate displacement prior to failure was 9.0 mm, giving a displacement ductility of 3.7.

The load-displacement response showed flexible behaviour after the general yielding of reinforcing bar. It was noted that the cracking of the concrete at the yield level was significant. The mode of the failure was due to splitting of the concrete along the tension leg and along the diagonal crack intersecting the end of reinforcing bar as shown in Figure 4.4(b). The variation of the strains along the length of the reinforcing bar at the ultimate load indicates that more than two-thirds of the tensile reinforcing bar leg had yielded (Figure 4.3). No yielding was noted in the compression leg.

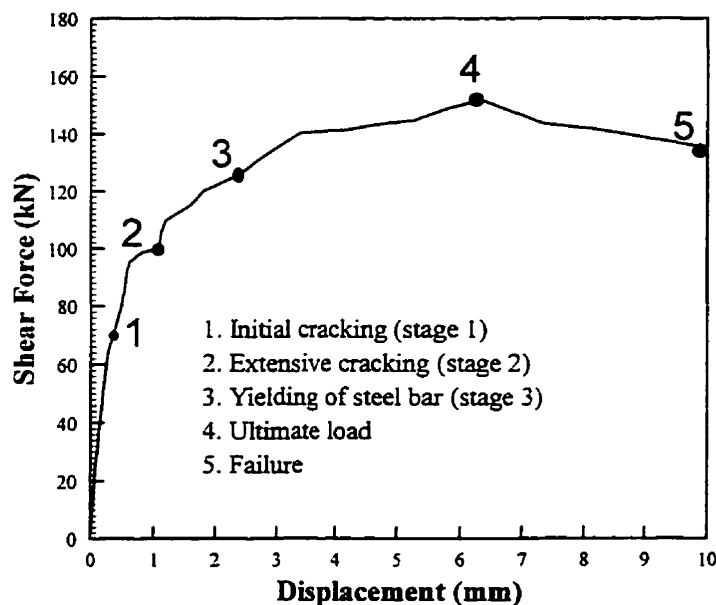
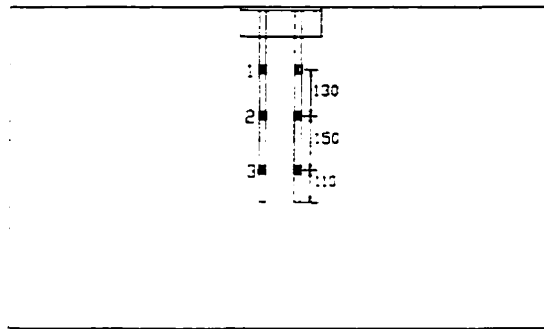
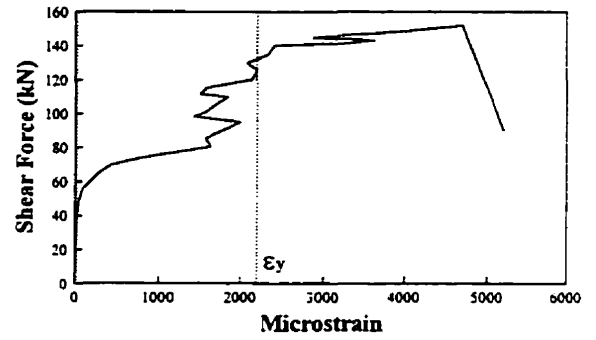


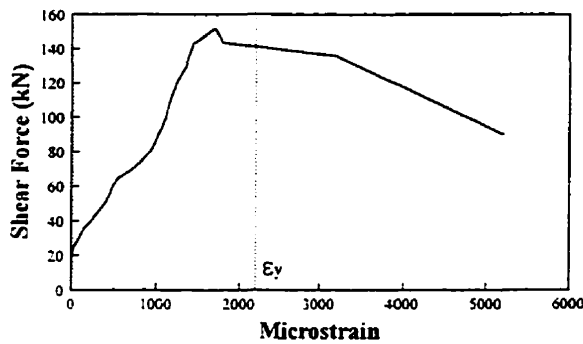
Figure 4.1 Shear force vs. horizontal displacement curve of Specimen CMS



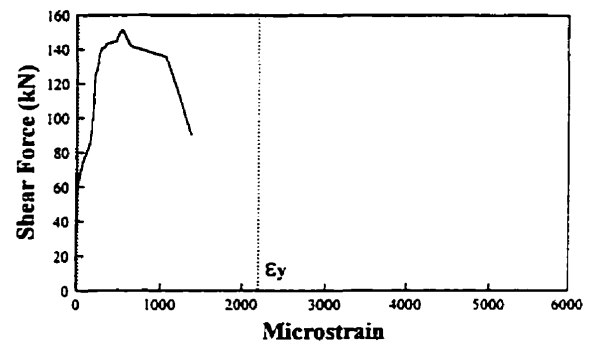
(a) Location of strain gauges



(b) Strains at Location 1



(c) Strains at Location 2



(d) Strains at Location 3

Figure 4.2 Measured strains along the length of anchor for Specimen CMS

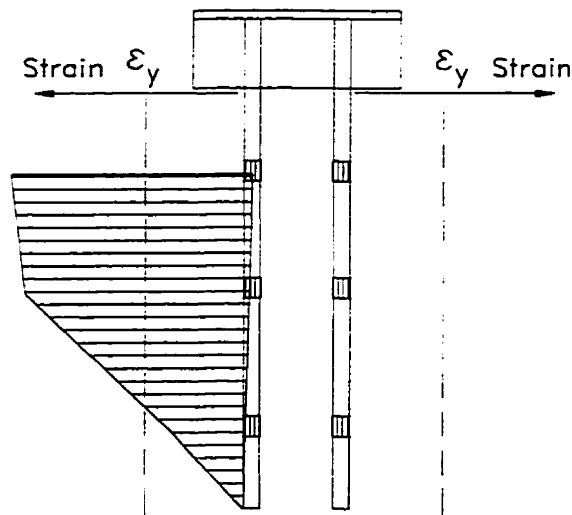
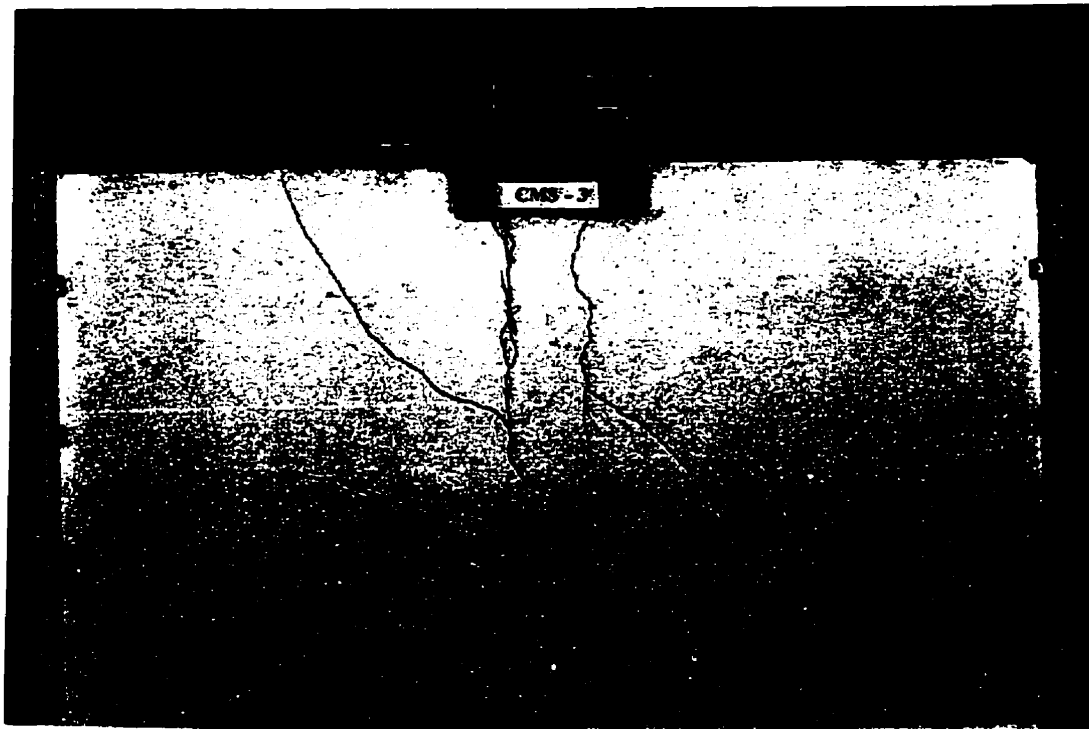
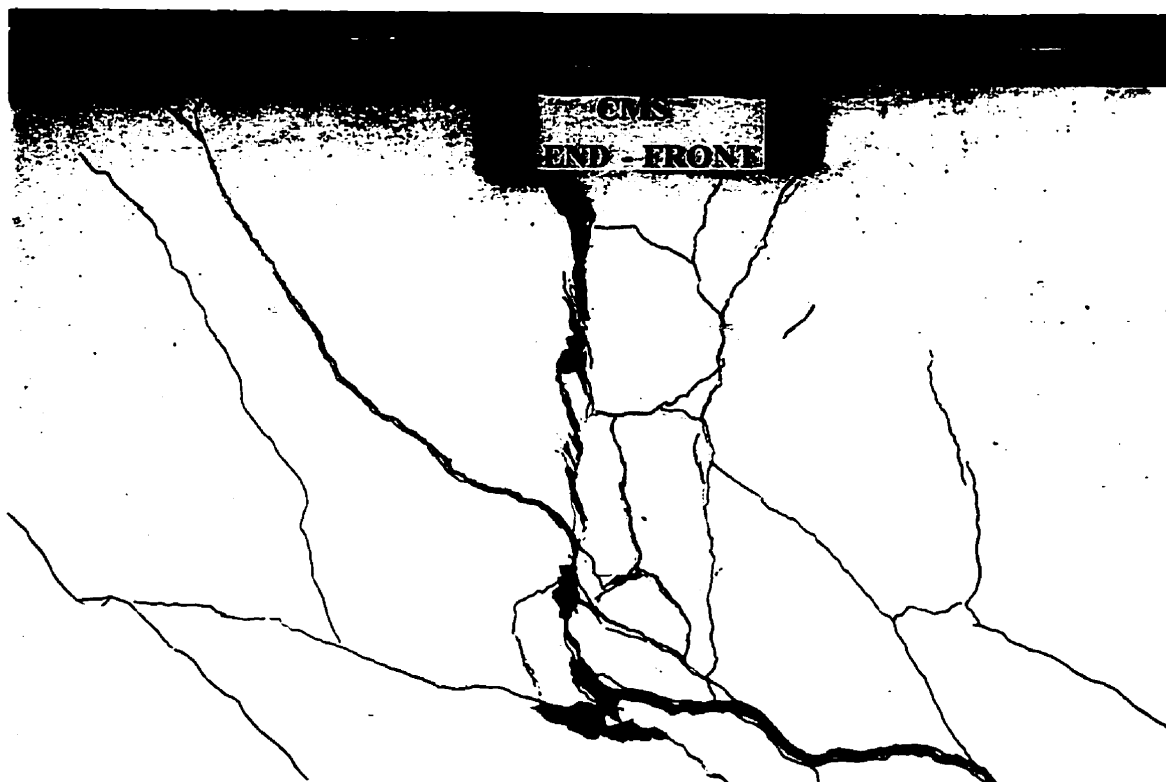


Figure 4.3 Strain distribution along the length of bar prior to failure of Specimen CMS



(a) Connection at first yield, Δ_y , $P=125$ kN (Stage 3)



(b) Connection after failure, $P_{ult}=135$ kN

Figure 4.4 Photographs of Specimen CMS

4.2.2 Specimen CS

The Specimen CS from The Series C was subjected to reversed cyclic shear only, as summarized in Table 4.1. The loading history and the hysteresis response of the specimen are presented in Figure 4.5. Figure 4.6 shows the shear force versus strain curves at locations along the anchor bars. The distribution of the strains with the distance along the length of reinforcing bar prior to failure is shown in Figure 4.7. Photographs of the specimen at the yield stage and after failure are shown in Figures 4.8(a) and 4.8(b), respectively. The following observation of the behavioural aspects were made during the testing and analysis of the experimental results.

Cycles 1-3, 38 kN No cracks appeared. The load-deflection response remained linear elastic up to end of this load stage.

Cycles 4-6, 75 kN The first crack appeared along both reinforcing bars at a load of 65 kN. This caused a change of slope in the load-deflection response (Figure 4.5), however, during the various cycles of this load stage, linear response was noted accompanied by almost negligible area within the hysteresis load-deflection loops, see Figure 4.5. The maximum crack width along the bar anchor was 0.3 mm. There was no notable energy dissipated up to the end of this stage.

Cycles 7-9, 105 kN The anchor bar yielded at Location 1 (Figure 4.6), resulting in a change in the stiffness of the connection and an increase in the displacement to $\Delta_y = 1.3$ mm (Figure 4.7). The splitting crack widened and extended along the entire length of the bar, while two diagonal cracks formed in the tension zone [see Figure 4.8(a)]. During the following cycles of this load stage, the loops were pinched (Figure 4.5) and the area under the loops was again not significant resulting in insignificant energy dissipation.

Cycles 10-12, 125 kN This stage involved use of displacement control, with maximum deflection $\Delta = 2\Delta_y = 2.6$ mm. The anchor bar yielded at Location 2 (Figure 4.6), indicating that yielding had developed along the bar anchor. This caused a significant decrease in the slope of load -deflection curve showing a reduction in the stiffness of the connection. No more cracks appeared, while the cracks were distributed all over the specimen. Due to the development of yielding and the crack propagation, the area under the loops increased during the following cycles. The hysteresis loops were pinched. The

crack width at the peak of the last cycle was 0.9 mm.

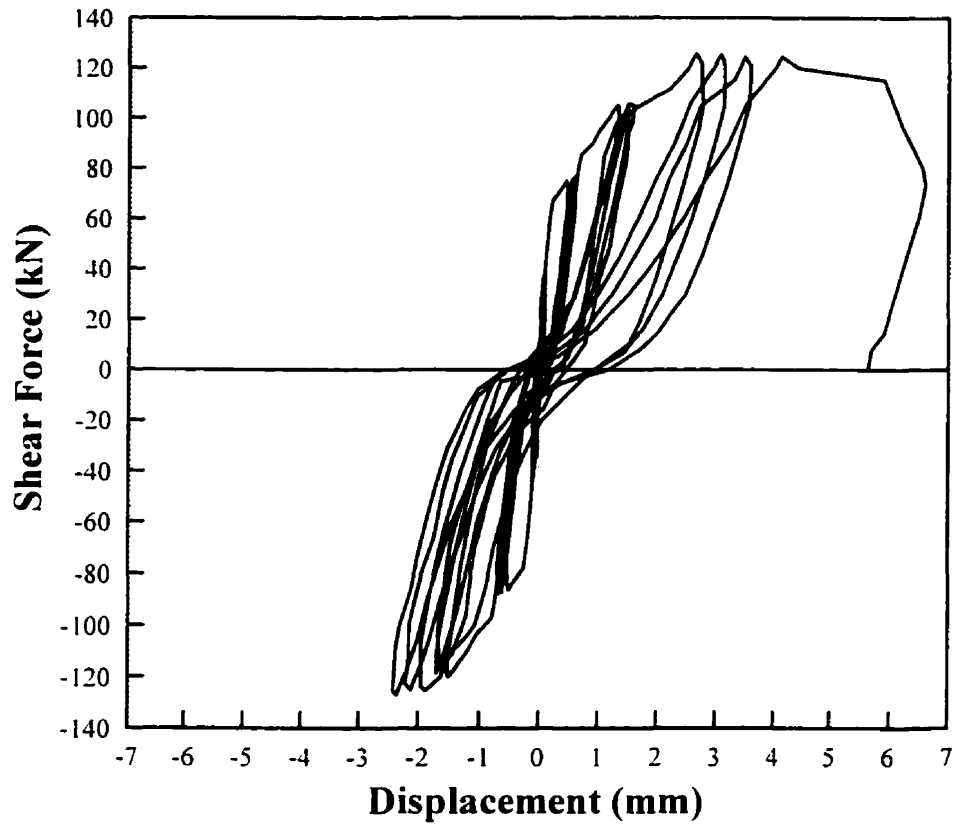
Cycle 13, 123 kN Finally, the specimen was subjected to a load of 123 kN, when the splitting and diagonal cracks opened more and the concrete between two legs started to crush and the stiffness of the connection (slope of load-deflection curve) kept on decreasing. The displacement increased significantly to about $5\Delta_y=6.6$ mm. The load then dropped to 75 kN, when the specimen failed. The connection could not resist any load in the other load direction, and the test was stopped. The area under the loops increased very significantly.

The variation of the strains along the anchor bar legs at the ultimate load shows that more than three-fourth of the tension leg of the anchor bar had yielded (Figure 4.7). The ultimate capacity achieved was 125 kN, while the final failure mode was the crushing-splitting of the concrete between the two legs close to the edge, and crushing-spalling of the concrete around the steel angle [see Figure 3.8(b)]. The hysteresis response consisted of large loops beyond the yielding of reinforcing bar, especially during the last cycle, showing the capability of connection to dissipate energy. The ultimate displacement prior to failure was 6.6 mm, giving a displacement ductility ratio of 5.

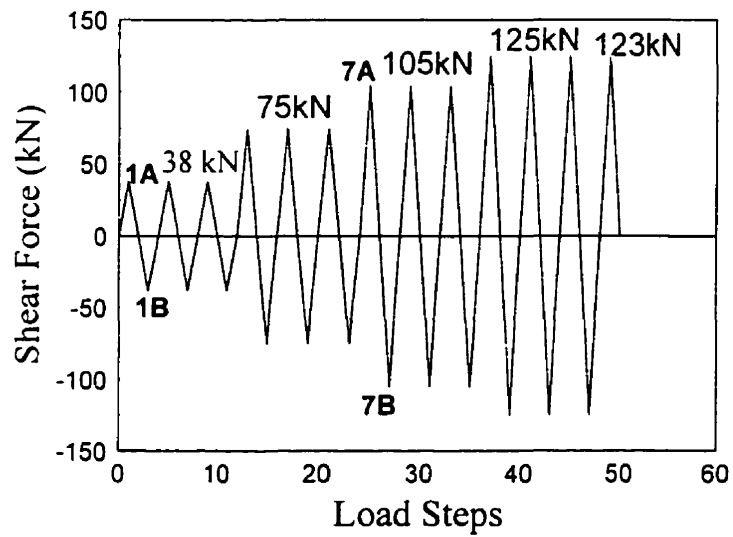
Table 4.1 Summary of Loading History for Specimen CS

Cycle	Load Stages	Horiz. Displacement (mm)*	Shear Force (kN)
Elastic	1A - 3B	+0.1 , -0.1	±38
Cracked	4A - 6B	+0.6 , -0.6	±75
±1 Δ_y	7A - 9B	+1.6 , -1.7	±105
±2 Δ_y	10A - 12B	+3.6 , -2.4	±125
Failure, +5 Δ_y	13A	+6.6	+123

* Maximum displacement at the peak load of the last cycle

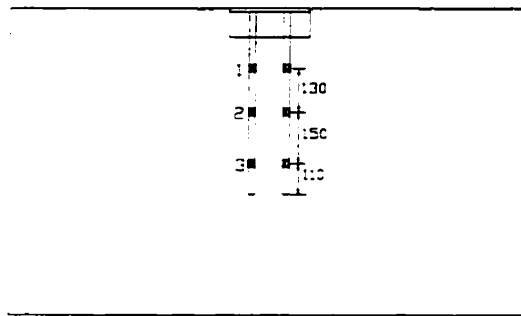


(a) Hysteresis response

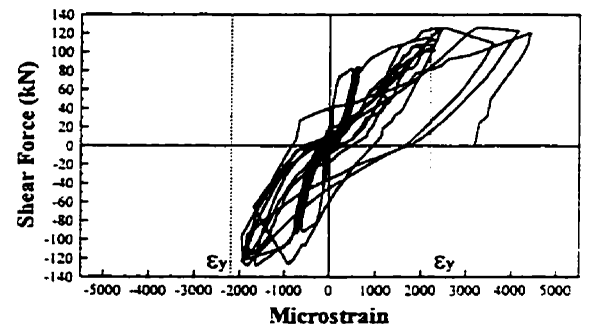


(b) Loading history

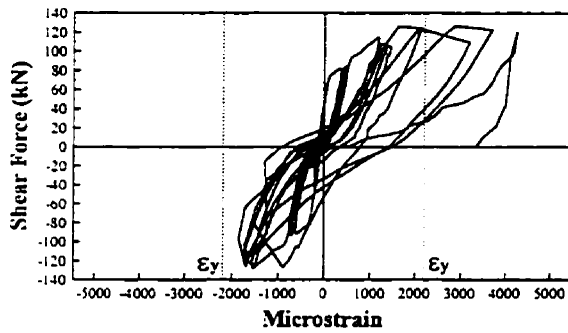
Figure 4.5 Hysteresis shear response and loading history for Specimen CS



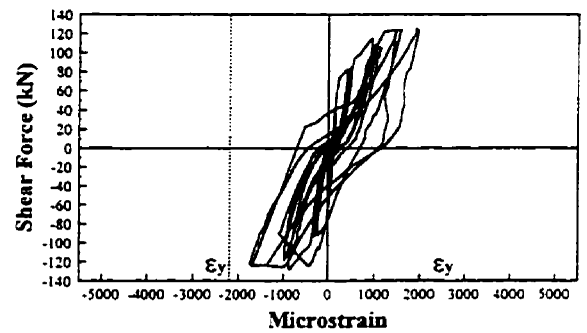
(a) Location of strain gauge



(b) Strains at Location 1



(c) Strains at Location 2



(d) Strains at Location 3

Figure 4.6 Measured strains along the length of anchor for Specimen CS

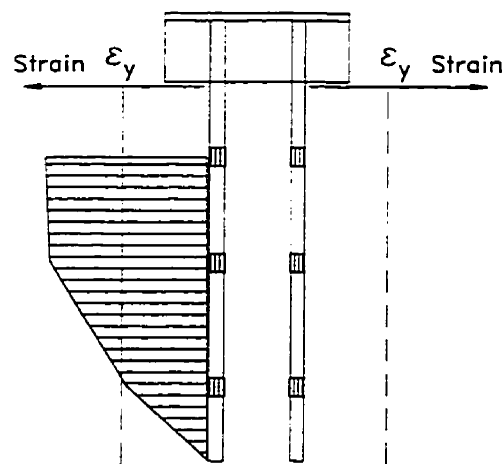
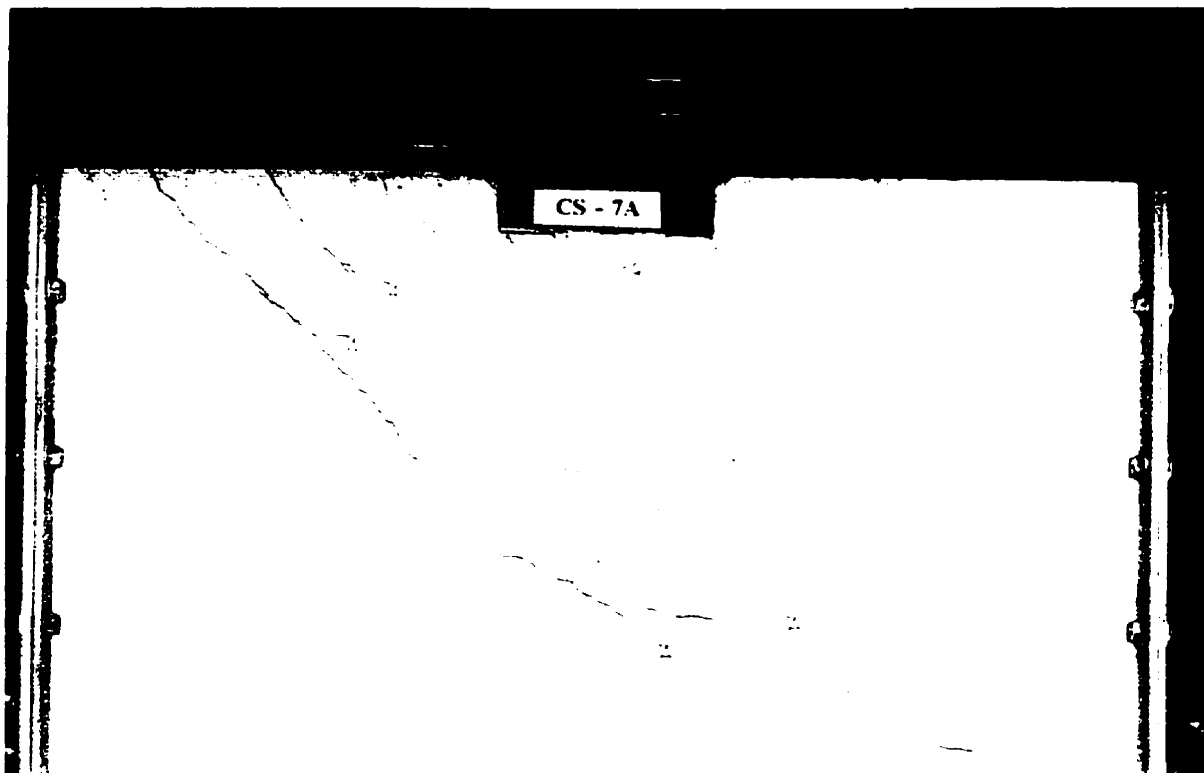
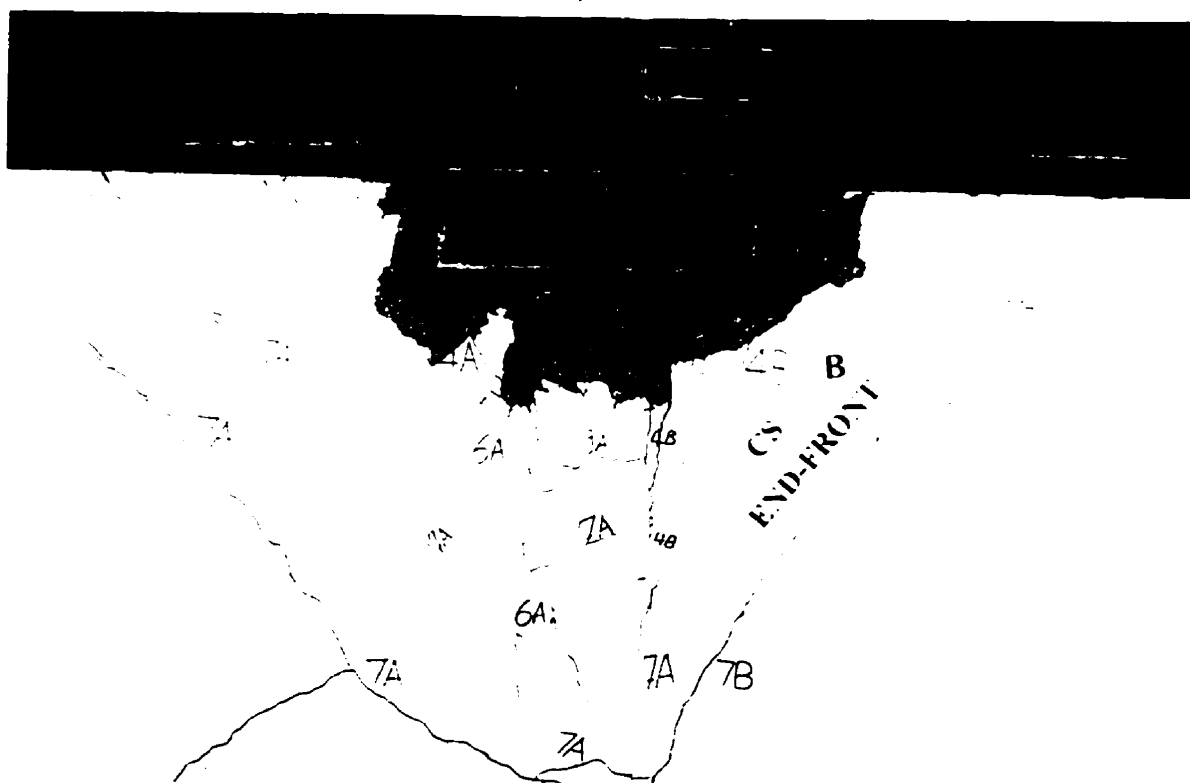


Figure 4.7 Strain distribution along the length of bar prior to failure of Specimen CS



(a) Connection at first yield, Δ_v , $P=105$ kN



(b) Connection after failure, $P_{ult}=132$ kN

Figure 4.8 Photographs of Specimen CS

4.2.3 Specimen CT

The Specimen CT was tested under reversed axial loads only. The loading history and the associated displacements are summarized in Table 4.2. Figure 4.9 shows the hysteresis response and the loading history of the specimen. The applied axial load versus the strain curves at the different locations along the reinforcing bar are shown in Figure 4.10. Figure 4.11 illustrates the distribution of the axial strain at the ultimate load versus the distance along the straight part of the anchor bar. Photographs of the connection at the yield stage, and after failure are presented in Figures 4.43(a) and 4.43(b), respectively. The following important behavioural aspects were noted during the testing and analysis of the experimental results.

Cycles 1-3, 50 kN The connection response during the first three cycles was basically linear elastic. No cracks appeared.

Cycles 4-6, 85 kN At load of 75 kN, the splitting crack appeared along both reinforcing bar legs, along with a diagonal crack close to edge. This caused reduction in the stiffness of the connection as manifested in the change in the slope of the load-deflection curve, however, during the following cycles of this load stage, the load-deflection response was elastic. The maximum splitting crack width measured was 0.15 mm. The displacements (Table 4.2) and the area under the hysteresis loops were small (Figure 4.9).

Cycles 7-9, 110 kN The reinforcing bar yielded at Location 1 at a load of 105 kN (Figure 4.10), resulting in a sharp change in the slope of the load-deflection curve and a decrease in the stiffness of the connection (Figure 4.9). The displacement at yield was $\Delta_y = 1.4$ mm. Splitting cracks extended along the length of the bar, while minor diagonal cracks and a major crack appeared between the bar legs and at the end of anchors, respectively, see Figure 4.12(a). During the following cycles (Cycles 11 and 12), the area under the hysteresis loops did not increase significantly.

Cycles 10-12, 140 kN This stage was implemented using displacement control, with the maximum deflection $\Delta = 2\Delta_y = 2.8$ mm. No more cracks appeared, while the diagonal and the splitting cracks widened further. The stiffness of the connection kept on decreasing, while the area under the loops kept on increasing. The maximum crack width

were 1.0 mm and 1.7 mm for the splitting and the diagonal cracks, respectively. The connection response was completely inelastic due to the extensive cracking and general yielding.

Cycles 13-15, 150 kN This stage was implemented using displacement control, with the maximum deflection equal to $3\Delta y = 4.2$ mm. The diagonal crack at the end of the reinforcing bar in tension opened to 2.0 mm. The steel angle moved axially (away from the specimen in the direction of the applied axial loads) and it detached from the concrete. The strains at Locations 2 and 3 were very close to the yield strain of the reinforcing steel bar. During the following cycles, although the applied load decreased to 140 kN, the displacements increased. This indicates a reduction in the connection strength and stiffness throughout the load cycles. The area under the hysteresis loops increased significantly, resulting in an increase in the energy dissipation. The maximum width of the diagonal and the splitting cracks were 2.2 mm and 3.1 mm, respectively.

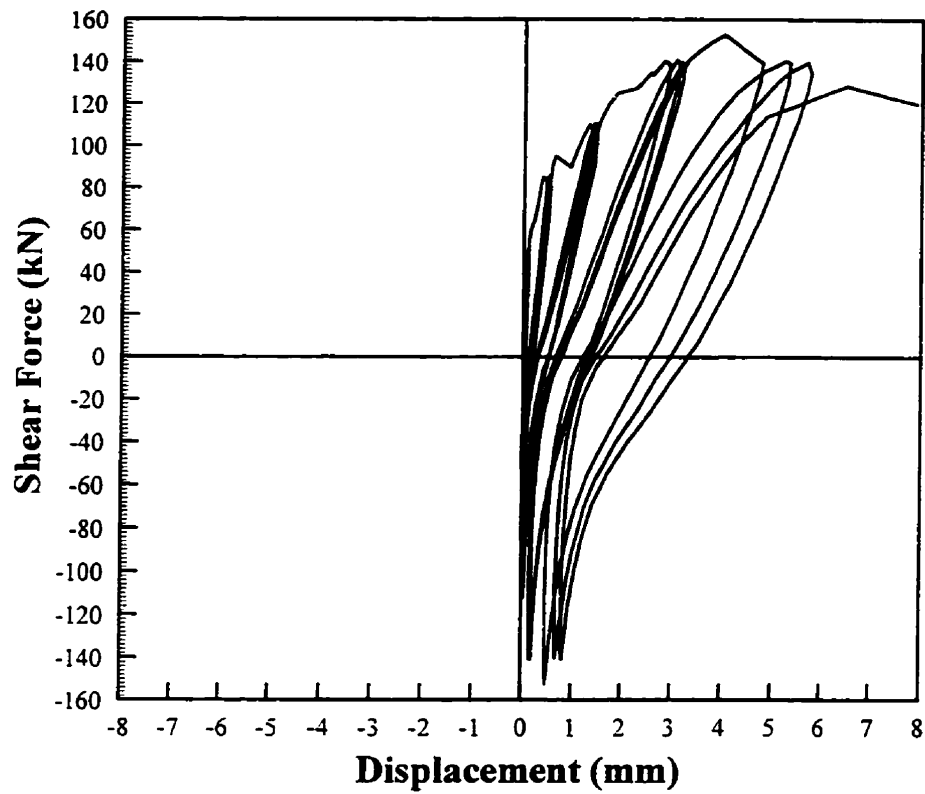
Cycle 16, 130 kN As the connection was subjected to a load of 130 kN, the displacement increased resulting in failure of the connection. The ultimate displacement prior to failure was 6.5 mm given displacement ductility ratio of 4.5.

The distribution of the axial strain along the bar anchor at the ultimate load (Figure 4.11) indicates that almost half of the reinforcing bar in tension had yielded. The maximum strength attained was 150 kN. The final mode of failure was due to the complete deterioration of the concrete surrounding the reinforcing bar and the steel angle [see Figure 4.12(b)]. The hysteresis response showed large loops in the positive direction (tensile load) which was due to the yielding of the reinforcing bar and the accompanying extensive cracking. Small loops in the negative direction (compressive load) were due to the imperfect closing of cracks during unloading of the tensile load and small concrete deformations when subjected to compressive stresses.

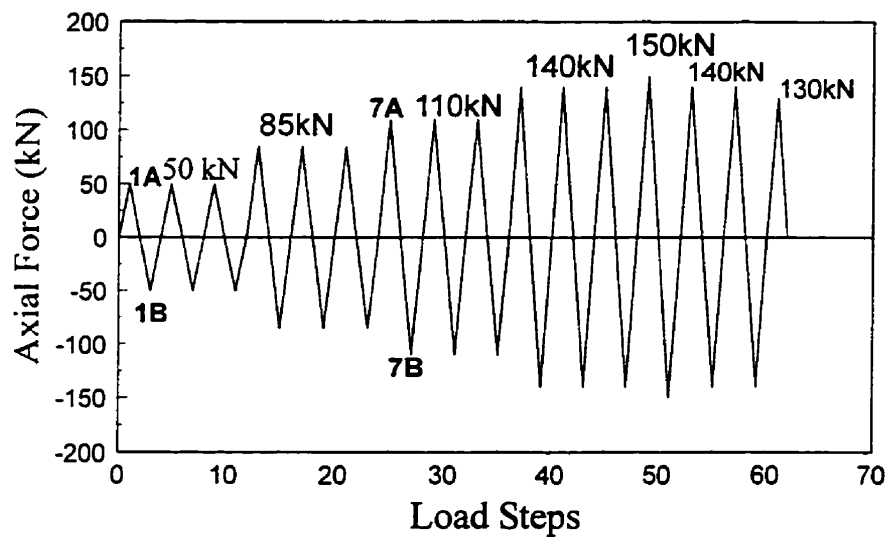
Table 4.2 Summary of Loading History for Specimen CT

Cycle	Load Stages	Axial Displacement (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.1 , -0.01	±50
Cracked	4A - 6B	+0.5 , -0.01	±85
±1Δy	7A - 9B	+1.5 , +0.03	±110
±2Δy	10A - 12B	+3.2 , +0.2	±140
±3Δy	13A - 15B	+5.7, +0.9	±150
Failure, 4.5Δy	16A	+6.5	+130

* Maximum displacement at the peak load of the last cycle

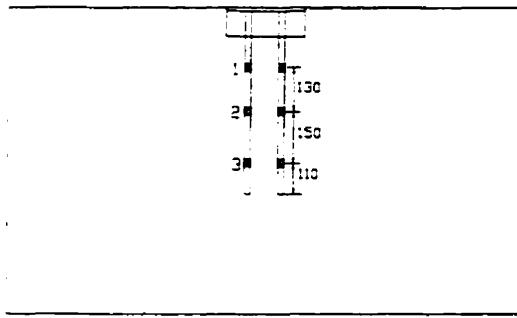


(a) Hysteresis response

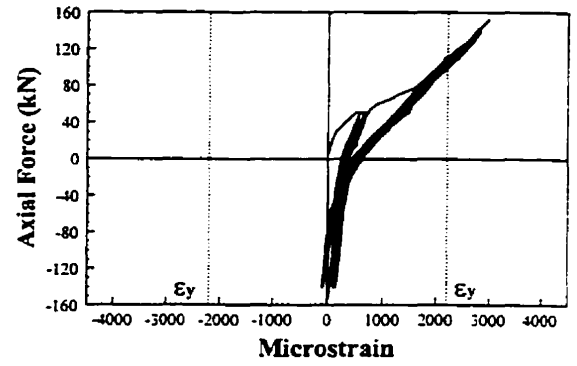


(b) Loading history

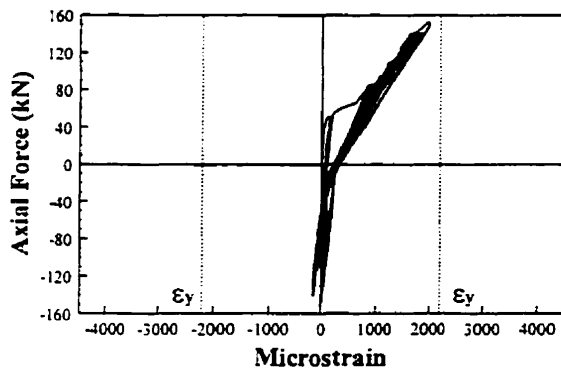
Figure 4.9 Hysteresis axial response and loading history for Specimen CT



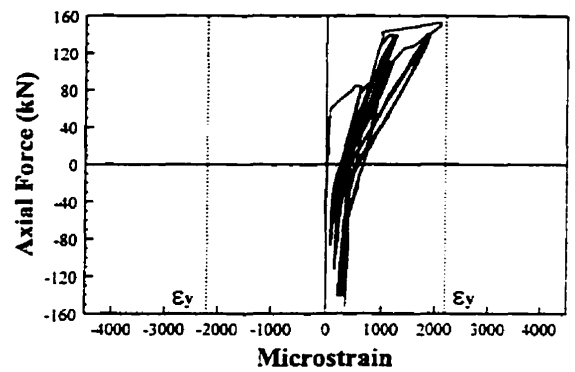
(a) Location of strain gauge



(b) Strains at Location 1



(c) Strains at Location 2



(d) Strains at Location 3

Figure 4.10 Measured strains along the length of anchor for Specimen CT

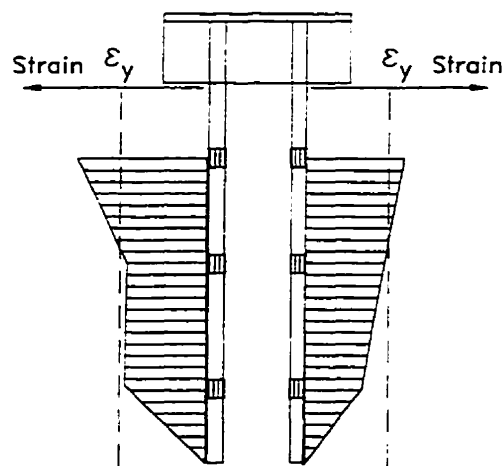
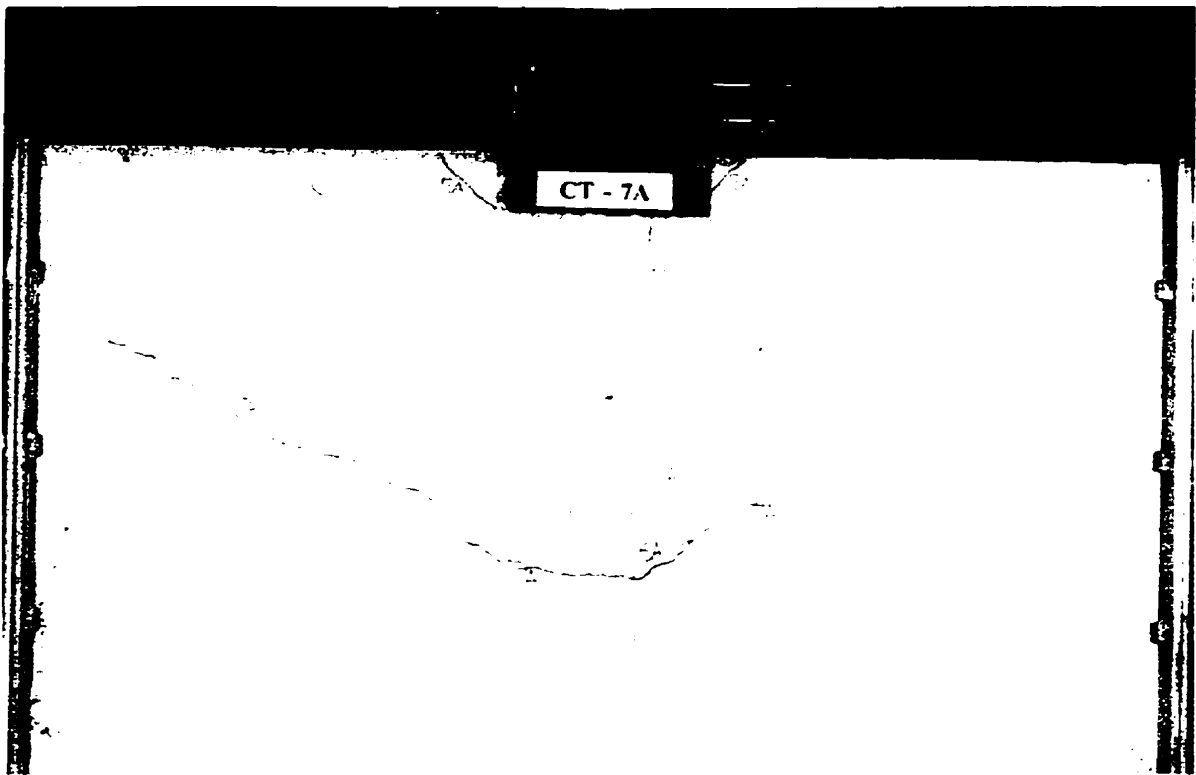
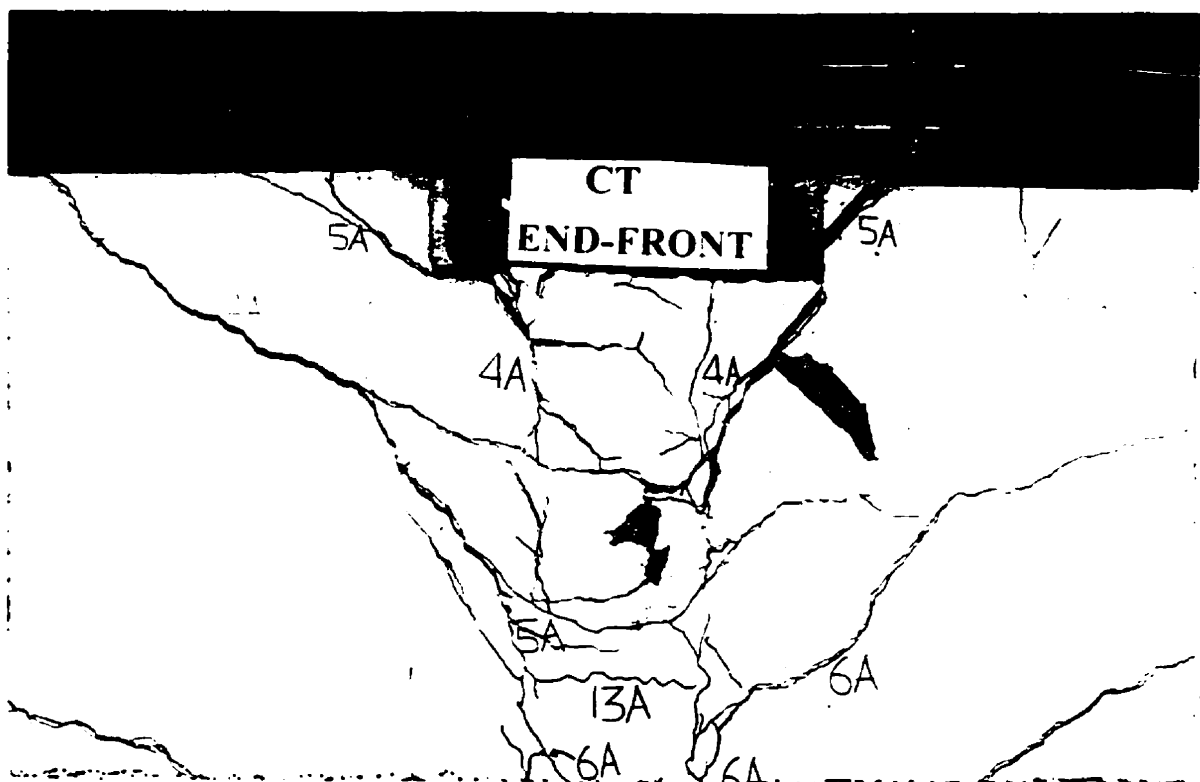


Figure 4.11 Strain distribution along the length of bar prior to failure of Specimen CT



(a) Connection at first yield, Δ_y , $P=110$ kN



(b) Connection after failure, $P_{ult}=130$ kN

Figure 4.12 Photographs of Specimen CT

4.2.4 Specimen CST1

The loading history and the resulting displacements for Specimen CST1 are summarized in Table 4.3. Figures 4.13 and 4.14 show the hysteresis responses and loading histories for the shear and axial loads (T/V ratio=0.7), respectively. The measured strains along the length of the reinforcing bar versus the shear and axial loads are presented in Figures 4.15 and 4.16. Figures 4.17(a) and 4.17(b) show the photographs of the specimen at the yielding stage and after failure, respectively. The distribution of strains with the distance along the length of the anchor bar at the ultimate load is shown in Figure 4.18. The following notes made based on the observation during the testing and analysis of the experimental results.

Cycles 1-3, $V=30$ kN $T=20$ kN The load-deflection response was linear elastic and no crack appeared up to the end of this load stage.

Cycles 4-6, $V=55$ kN $T=40$ kN Small splitting crack along the tension leg and a diagonal crack appeared in tension zone. The slope of the load deflection curve changed slightly and nonlinearity started (Figures 4.13 and 4.14). The displacements were very small (Table 4.3).

Cycles 7-9, $V=80$ kN $T=55$ kN The reinforcing bar anchor yielded at Location 1 (Figures 4.15 and 4.16). The measured horizontal and axial displacements (Δ_y) were 1.8 mm and 1.9 mm, respectively. Due to the yielding of the steel bar, significant diagonal cracking in tension zone and the splitting cracking along the compression leg were noted [Figure 4.17(a)]. This caused a reduction in the connection stiffness (Figures 4.13 and 4.14). The maximum width of the splitting and diagonal cracks were 0.5 mm and 0.9 mm, respectively. The area under the hysteresis loops increased during the positive load (shear and axial tension), however in the negative load direction (shear and axial compression), the area within the loops was very small. The dissipated energy was considerable.

Cycles 10-12, $V=100$ kN $T=65$ kN This stage was implemented using displacement control, with maximum deflection equal to $2\Delta_y$. The horizontal and axial displacements were 3.8 mm and 4.9 mm, respectively. The reinforcing bar yielded at Location 2, which indicates the extension of yielding along the tension steel anchor leg

(Figures 4.15 and 4.16). During the following cycles of this stage, as the displacement was maintained of about $2\Delta_y$, the axial and shear loads both decreased [Figures 4.13(a) and 4.14(b)], which indicates a decrease in the stiffness and strength of the connection due to cyclic loading. The area within the hysteresis loops increased, especially in the positive direction, showing the capability of the connection to dissipate energy throughout development of yielding along the reinforcing bar and the cracking of the specimen. By the end of this load stage, the maximum widths of the splitting and diagonal cracks were 2.1 mm and 1.7 mm, respectively. During the last cycle, the displacements increased significantly (Table 4.3) resulting in an increase in the area under the hysteresis loops.

Cycles 13-15, $V=85 \text{ kN}$ $T=55 \text{ kN}$ This stage involved use of displacement control, with a maximum deflection $\Delta=3\Delta_y$. The horizontal and axial displacements were 4.9 mm and 6.2 mm, respectively. During the following cycles at this load stage, the stiffness of the connection and the applied shear and axial loads decreased, while the displacements increased significantly (Table 4.3). The area under the hysteresis loops increased considerably, indicating significant energy dissipation throughout the cyclic loading. The width of cracks increased significantly, especially for the splitting crack along the tension steel anchor leg. The maximum splitting crack width was 3.0 mm. The concrete around the steel angle spalled off, allowing the angle to move easily. The measured horizontal and axial displacements at the end of this stage were 11.8 mm and 10.2 mm, respectively (Table 4.3).

Cycle 16, $V=50 \text{ kN}$ $T=30 \text{ kN}$ The specimen underwent large displacement and finally failed. The final mode of failure was the splitting bond failure and spalling of the concrete along the tension leg plus crushing-spalling of the concrete surrounding the steel angle [Figure 4.17(b)]. The ultimate horizontal and axial displacements prior to failure were measured to be 12.9 mm and 12.4 mm, respectively.

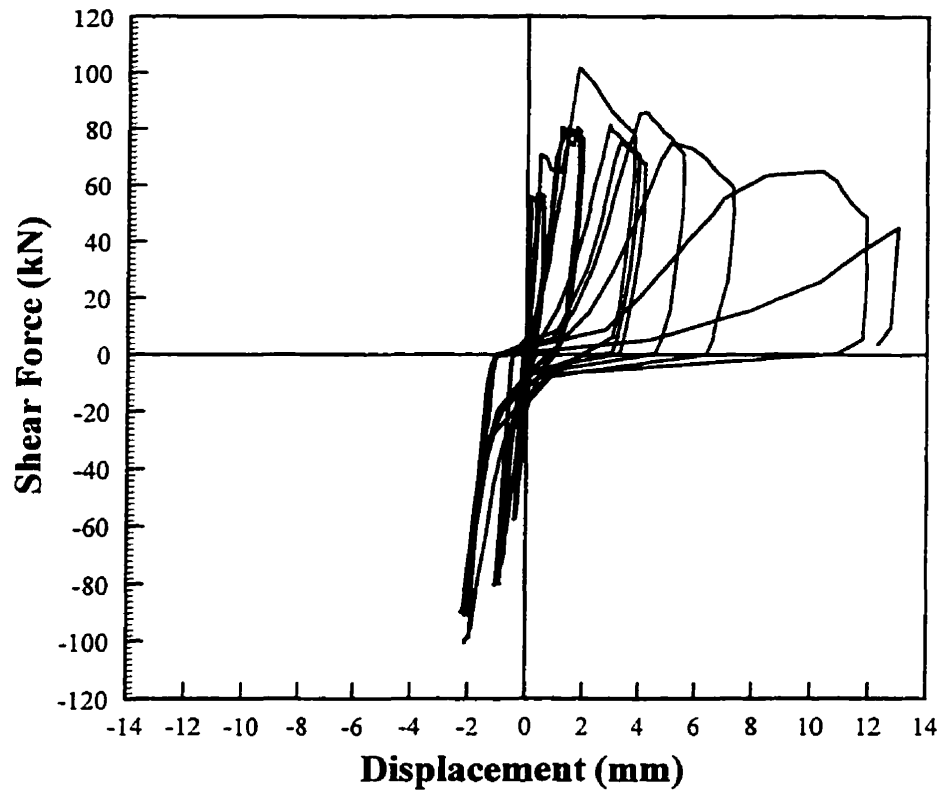
The distribution of the strain along the anchor bar legs at the ultimate load show that about four-fifths of the reinforcing bar in the tension leg had yielded (Figure 4.18). The maximum shear and axial strengths of 100 kN and 65 kN were attained. the last cycle of loading (Cycle 16) cannot be considered to be stable loops, therefore the displacements at the end of Cycle 15 were used to calculate the displacement ductility,

which gave a displacement ductility ratio of 6. The results show that under combined cyclic axial and shear loads ($T/V=0.7$), the strength of the connection decreased to about 80 % and 43% of those for pure cyclic shear and pure cyclic axial strengths, respectively. The hysteresis response showed larger loops in the positive direction which was due to the tensile axial load and development of yielding along the reinforcing bar. The smaller loops in the negative direction were due to the compressive axial loads, imperfect closing of the cracks and small deformability of the concrete subjected to the compressive stresses. The hysteresis loops were pinched which is an indicator of the reduction in dissipated energy.

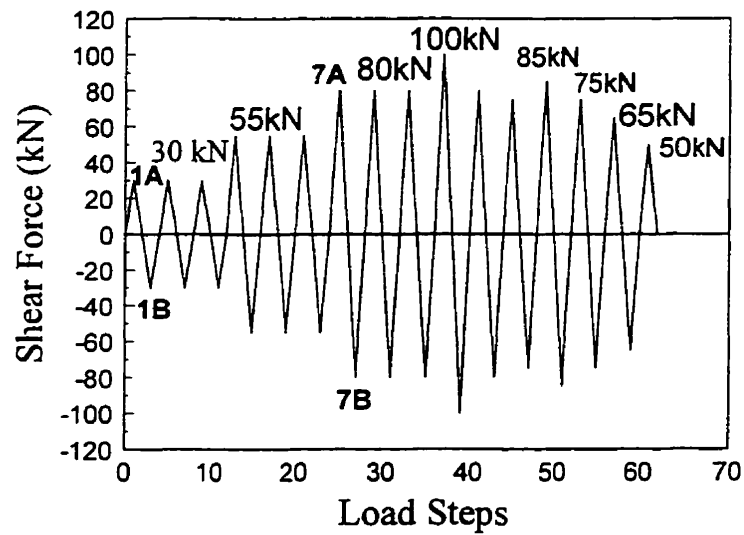
Table 4.3 Summary of Loading History for Specimen CST1

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.1 , -0.1	±30	+0.2 , +0.01	±20
Cracked	4A - 6B	+0.6 , -0.4	±55	+0.7 , -0.01	±40
±1 Δy	7A - 9B	+1.9 , -1.0	±80	+2.2 , +0.2	±55
±2 Δy	10A -12B	+4.2 , -2.1	±100	+5.0 , +0.6	±65
±3 Δy	13A -14B	+7.3 , -1.9	±85	+6.6 , +0.7	±55
±6 Δy	15A - 15B	+11.8 , -1.8	±65	+10.2 , +0.8	±35
Failure	16A	+12.9	+50	+12.4	+30

* Maximum displacement at the peak load of the last cycle

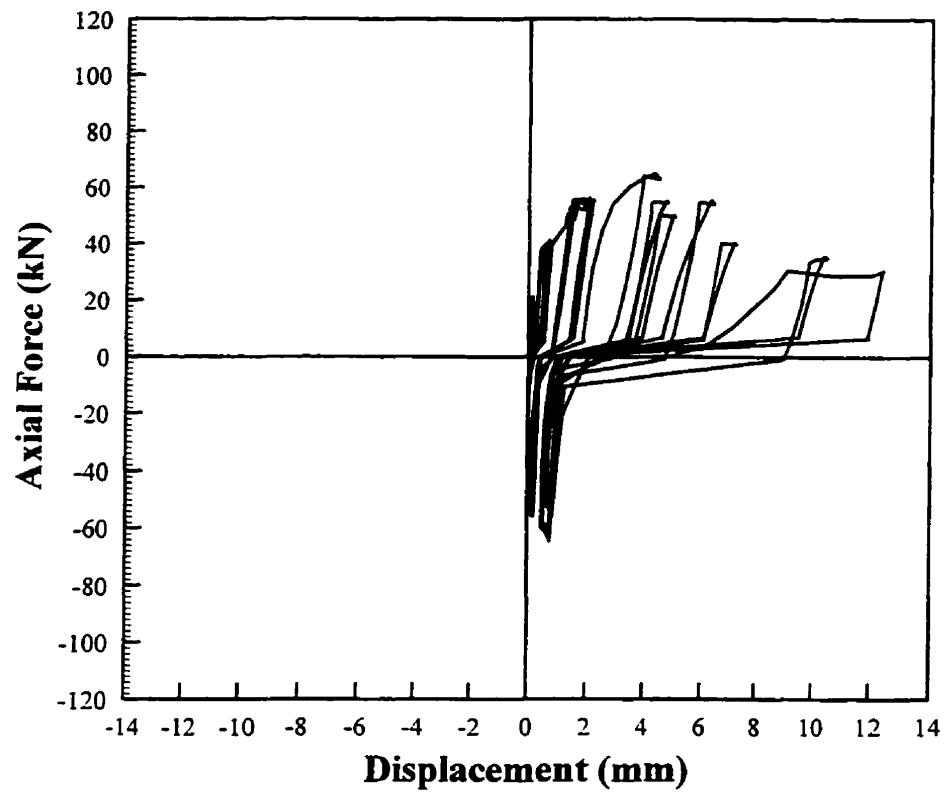


(a) Hysteresis response

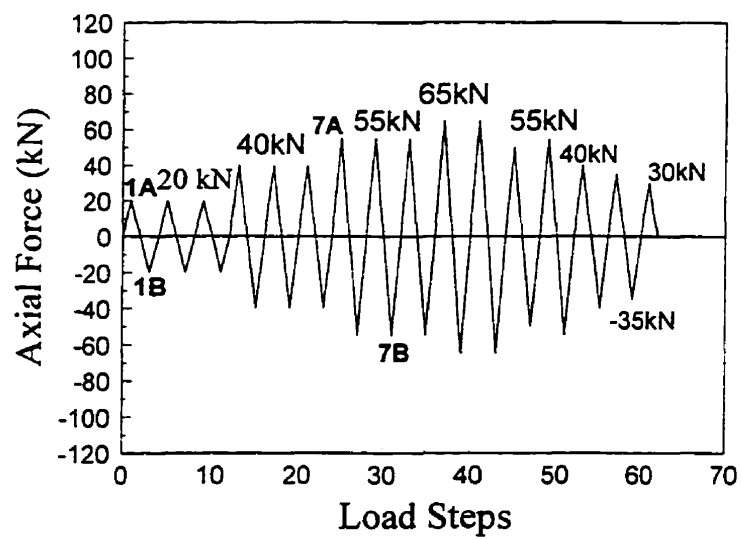


(b) Loading history

Figure 4.13 Hysteresis shear response and loading history for Specimen CST1

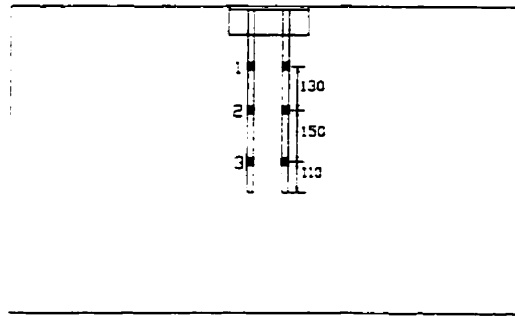


(a) Hysteresis response

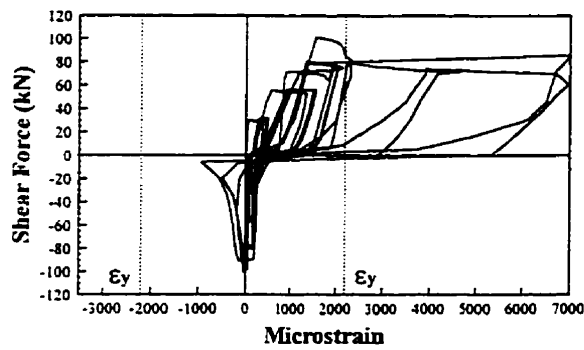


(b) Loading history

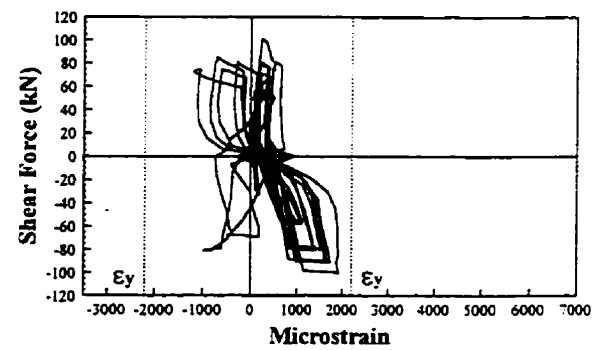
Figure 4.14 Hysteresis axial response and loading history for Specimen CST1



(a) Location of strain gauge

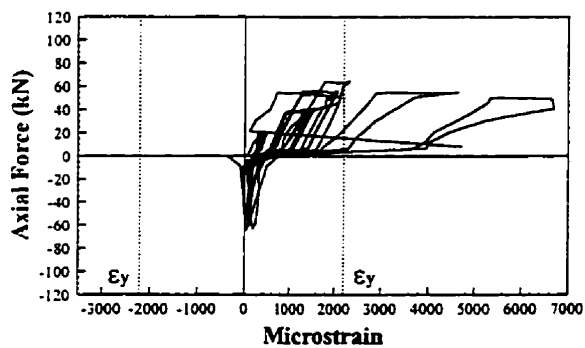


(b) Strains at Location 1, Shear response

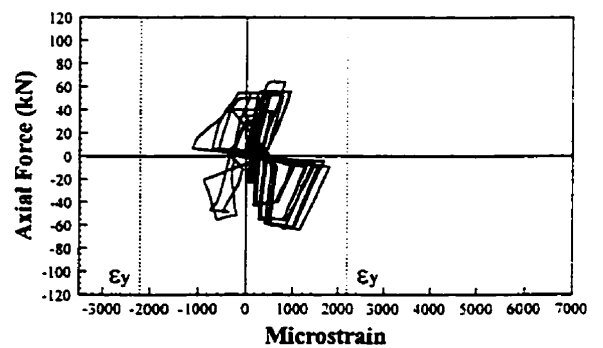


(c) Strains at Location 4, Shear response

Figure 4.15 Strains along length of anchor in Specimen CST1 (Shear response)

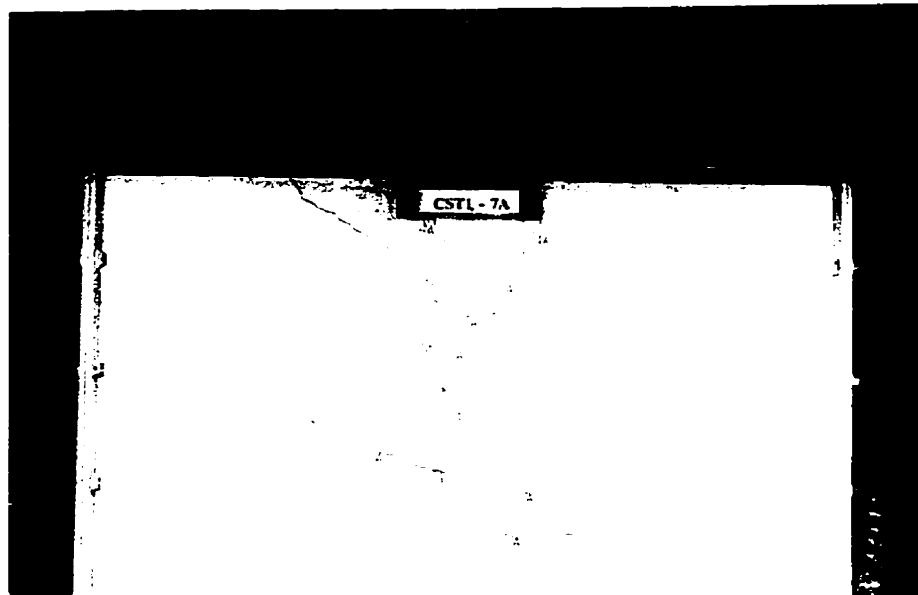


(b) Strains at Location 1, Axial response

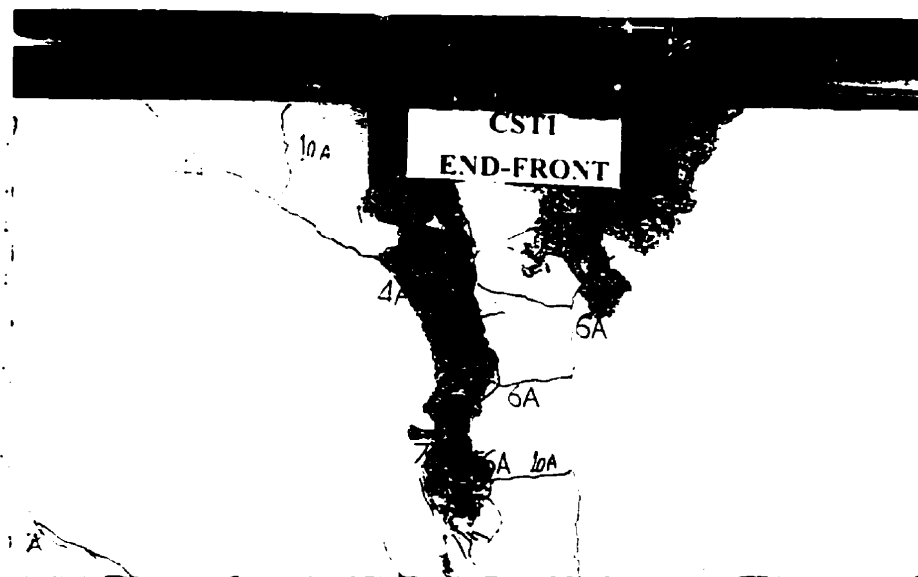


(c) Strains at Location 4, Axial response

Figure 4.16 Strains along length of anchor in Specimen CST1 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}}=80 \text{ kN}$, $P_{\text{axial}}=55 \text{ kN}$



(b) Connection after failure, $P_{\text{ult, shear}}=65 \text{ kN}$, $P_{\text{ult, axial}}=35 \text{ kN}$

Figure 4.17 Photographs of Specimen CST1

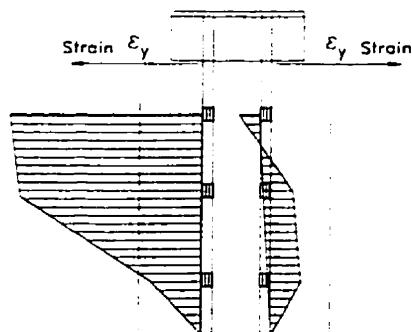


Figure 4.18 Strain distribution along the length of bar prior to failure of Specimen CST1

4.2..5 Specimen CST2

The loading history and the associated displacements for Specimen CST2 are presented in Table 4.4. The load-deflection response and the loading histories for the shear and axial loads (T/V ratio = 1.0) are shown in Figures 4.19 and 4.20, respectively. The strains along the length of the steel anchor bar for shear and axial loads are shown in Figures 4.21 and 4.22, respectively. The profile of the axial strain with the distance along the length of anchor bar at the ultimate load are shown in Figure 4.24. The photographs of the specimen at the yielding level and after failure are shown in Figures 4.23(a) and 4.23(b), respectively. The following important behavioural aspects were noted during the testing and from the analysis of the experimental results:

Cycles 1-3, $V=T=25$ kN The hysteresis response was linear elastic without any cracking occurring during these load cycles.

Cycles 4-6, $V=T=50$ kN Small splitting crack appeared along the tension leg and one diagonal crack was noted in the tension zone. Due to cracking, the slope of the load-deflection curves changed and nonlinear response started. During Cycle 6, another diagonal crack was observed in the tension zone. The maximum width of the splitting and the diagonal cracks at the end of this stage were 0.15 mm and 0.2 mm, respectively. The horizontal (shear) and vertical (axial) displacements were 0.94 mm and 1.2 mm, respectively (Table 4.4). The hysteresis loops were pinched (Figures 4.19 and 4.20). The area under the loops was small showing insignificant dissipated energy.

Cycles 7-9, $V=T=70$ kN The first yielding occurred at Location 1 (Figures 4.21 and 4.22). The measured horizontal and axial displacements (Δ_y) were 1.4 mm and 2.1 mm, respectively. Diagonal and splitting cracks in the tension zone extended toward the specimen centre and along the entire length of anchor bar, respectively [Figure 4.23(a)]. Due to the yielding of the steel anchor, the stiffness of the connection decreased and the area under the hysteresis loops increased. The energy dissipation after the yielding stage was considerable. At the end of this stage, the maximum splitting and diagonal crack widths were 1.0 mm each. During the following cycles of this loading stage, no more cracks appeared.

Cycles 10-12, $V=T=80$ kN This stage was implemented using displacement control with

maximum deflection equal to $2\Delta_y$. The horizontal and the axial displacements were 3.2 mm and 3.7 mm, respectively. A splitting crack appeared along the compression leg, while some horizontal cracks occurred between the two legs due to the flexural behaviour of the concrete between these two reinforcing bar. During the following cycles of this load stage, no major changes were noted. The area under the hysteresis loops increased, especially in the positive direction, showing the capability of connection to dissipate energy through development of yielding along the reinforcing bar, propagation of cracks and crack widening over the specimen. The steel strains at Locations 1 and 4 increased, while it did not change at other locations (Figures 4.21 and 4.22). The maximum measured splitting and diagonal crack widths were 2.0 mm, each .

Cycles 13-15 $V=90$ kN $T=85$ kN At this stage, the maximum displacement was $3\Delta_y$. The shear and axial displacements were 5.5 mm and 5.6 mm, respectively. The reinforcing bar at Location 4, which was subjected to compressive strain due to dowel action, yielded (Figures 4.21 and 4.22). During the following cycles of this load stage, the stiffness of the connection decreased significantly. The area under the loops for both shear and axial responses increased significantly resulting in considerable energy dissipation throughout the cycling loading (Figures 4.19 and 4.20). During the last cycle, in order to keep the displacement around $3\Delta_y$, the shear and axial loads were reduced to 85 kN and 70 kN, respectively. This indicates a significant reduction in the stiffness of the connection. By the end of this stage, the cracks extended and opened significantly, and the splitting and diagonal crack widths were 4 mm and 3.5 mm, respectively.

Cycles 16-17 $V=85$ kN $T=60$ kN Because of excessive deterioration of the concrete during the last stage, an axial load of 60 kN was applied simultaneously with a shear force of 85 kN. The stiffness of the connection decreased, and the displacement increased significantly, resulting in a gradual decrease of the shear load to about 40 kN. The area under the hysteresis loops increased significantly despite the reduction in the shear force. The horizontal and the axial displacements were measured to be 10.6 mm and 11.9 mm, respectively. This indicates a significant decrease in the strength and stiffness of the connection due to the reversed cyclic loading and application of the axial load. During the last cycle, as shear and axial load of 40 kN and 35 kN were applied, the concrete in

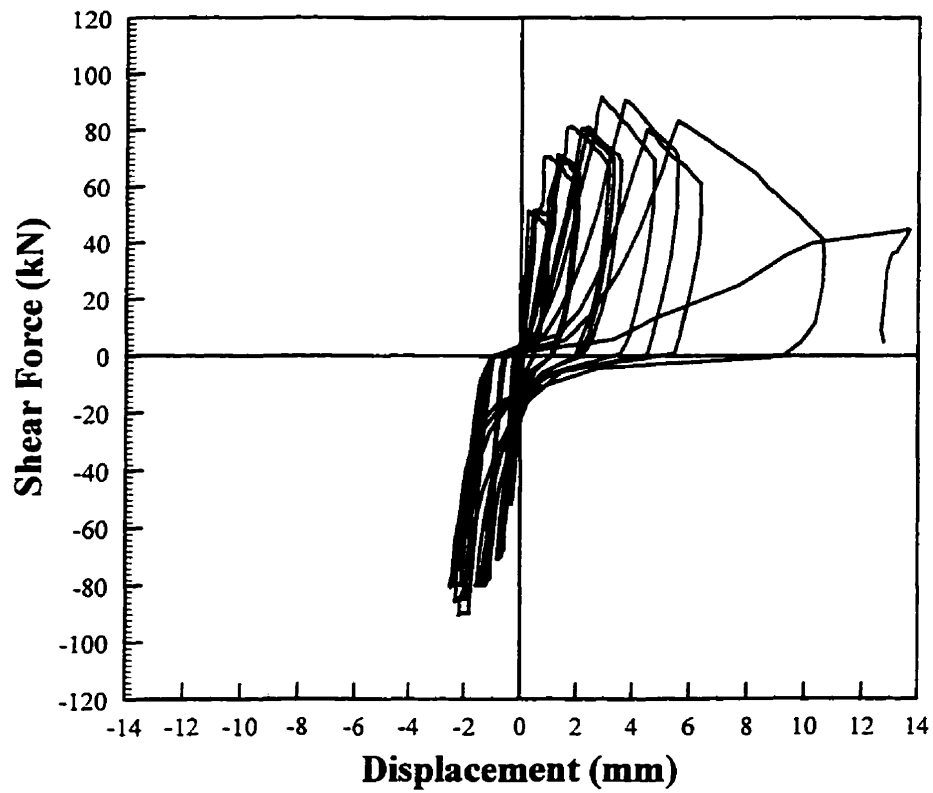
the tension zone cracked completely [Figure 4.23(b)], and specimen underwent large displacements (Figures 4.19 and 4.20). The connection was unloaded and the test was terminated. The last cycle cannot be considered to represent stable hysteresis loops, therefore the displacements during previous cycle had to be used to calculate the connection ductility, giving the displacement ductility, ratio of 6.3.

The distribution of the strain along the anchor leg at the ultimate load shows that about one-half of the length of reinforcing bar in tension had yielded (Figure 4.24). Maximum shear and axial strengths of 90 kN and 85 kN were achieved. The final mode of failure was due to complete bond failure along the tension leg (splitting) along with the crushing of the concrete around the steel angle [Figure 4.23(b)]. The failure mode was close to that for the Specimen CST1. Based on the experimental results, under combined cyclic axial and shear loads ($T/V=1.0$), the strength of connection reduced to about 72% and 57% of those for pure cyclic shear and pure cyclic axial strengths, respectively. The hysteresis response showed large loops, especially beyond the general yielding, and opening of the splitting and the diagonal cracks. The displacement ductility ratio attained at the ultimate load was 6.3.

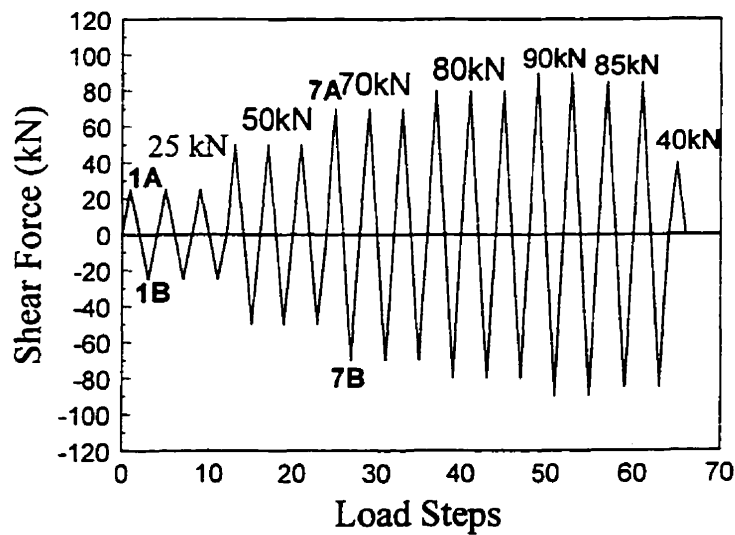
Table 4.4 Summary of Loading History for Specimen CST2

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.2 , -0.1	±25	+0.3 , +0.03	±25
Cracked	4A - 6B	+0.9 , -0.4	±50	+1.2 , +0.1	±50
±1 Δy	7A - 9B	+2.1 , -0.9	±70	+2.5 , +0.2	±70
±2 Δy	10A - 12B	+3.5 , -1.5	±80	+4.2 , +0.4	±80
±3 Δy	13A -15B	+6.4 , -2.5	±90	+8.4, +0.9	±85
6.3 Δy	16A -16B	+10.6, -2.4	±85	+10.8, +1.0	±60
Failure	17A	+13.6	+40	+11.9	+35

* Maximum displacement at the peak load of the last cycle

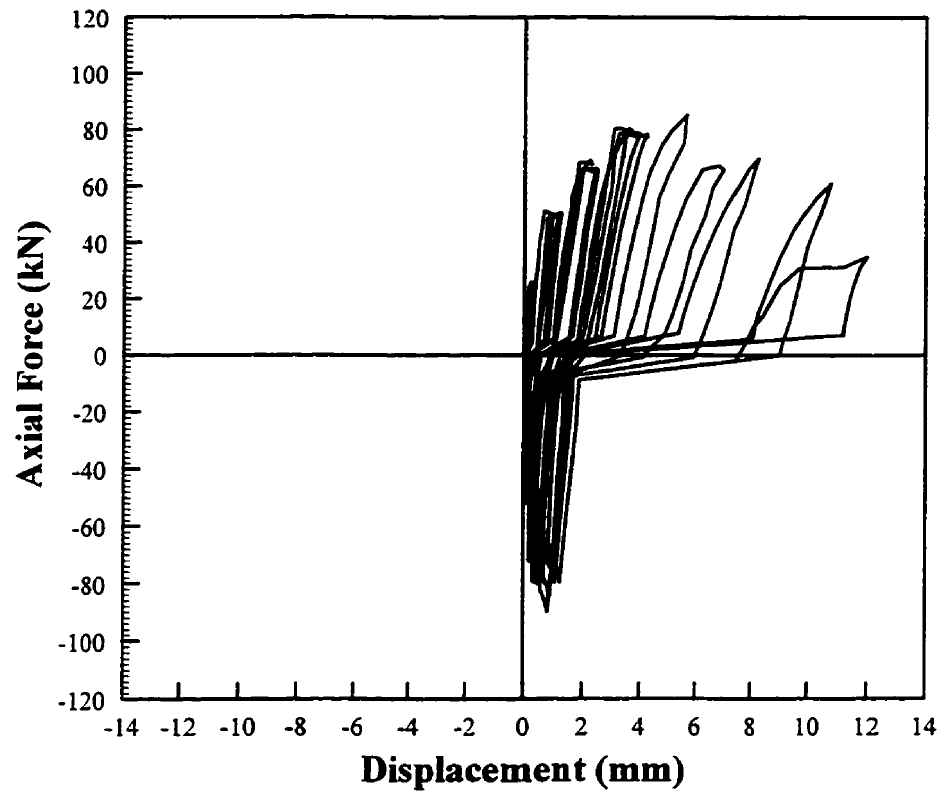


(a) Hysteresis response

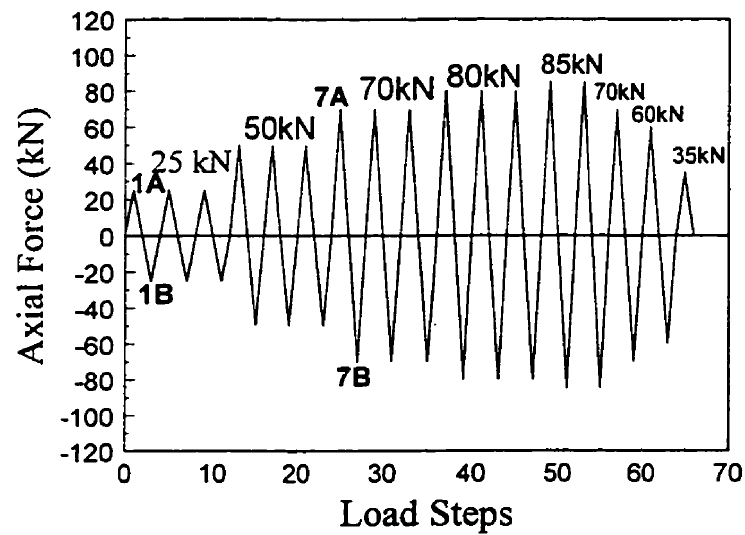


(b) Loading history

Figure 4.19 Hysteresis shear response and loading history for Specimen CST2

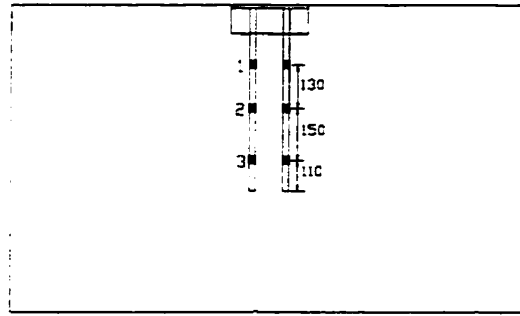


(a) Hysteresis response

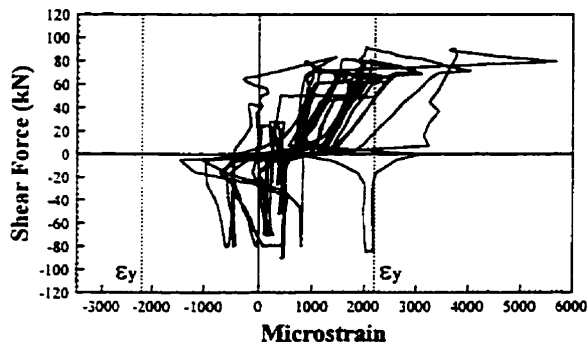


(b) Loading history

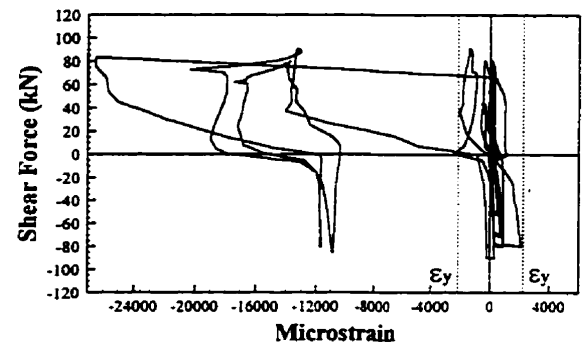
Figure 4.20 Hysteresis axial response and loading history for Specimen CST 2



(a) Location of strain gauge

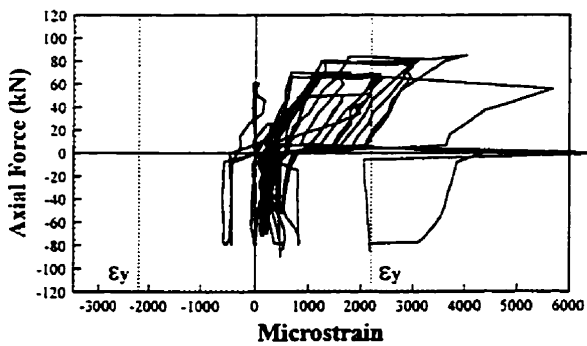


(b) Strains at Location 1, Shear response

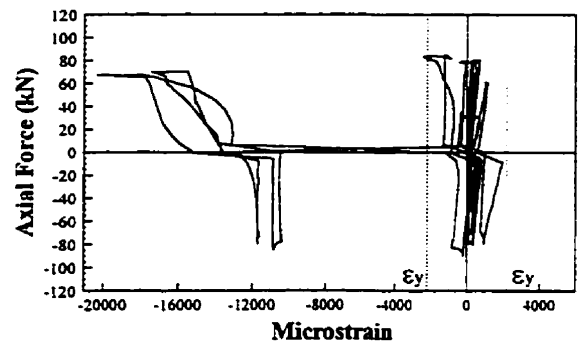


(c) Strains at Location 4, Shear response

Figure 4.21 Strains along length of anchor in Specimen CST2 (Shear response)

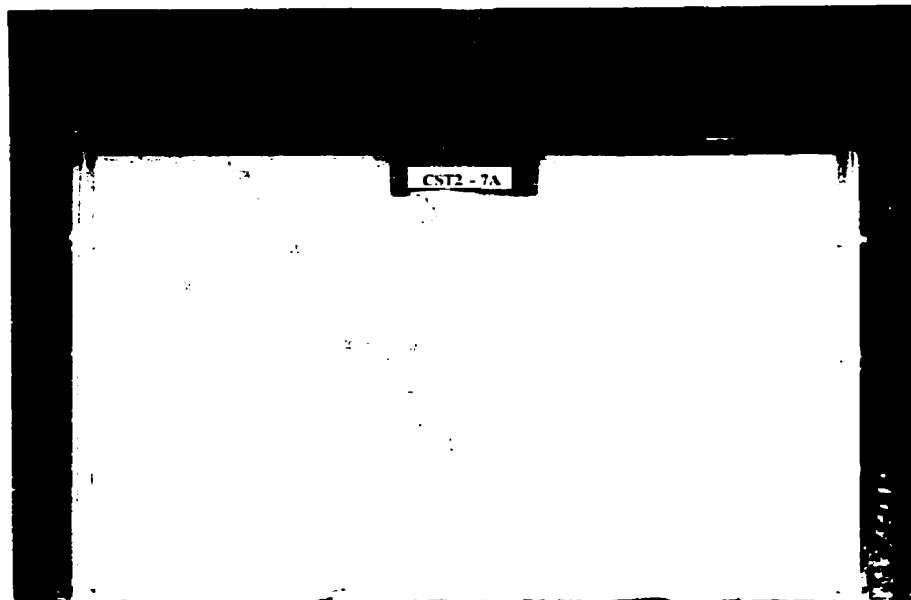


(b) Strains at Location 1, Axial response



(c) Strains at Location 4, Axial response

Figure 4.22 Strains along length of anchor in Specimen CST2 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}} = P_{\text{axial}} = 70 \text{ kN}$



(b) Connection after failure, $P_{\text{ult, shear}} = 85 \text{ kN}$, $P_{\text{ult, axial}} = 60 \text{ kN}$

Figure 4.23 Photographs of Specimen CST2

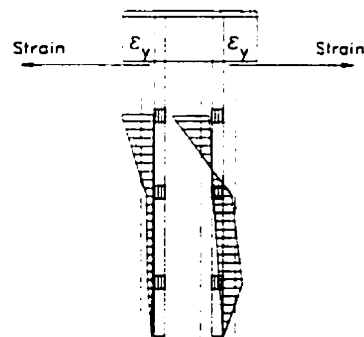


Figure 4.24 Strain distribution along the length of bar prior to failure of Specimen CST2

4.2.6 Specimen CST3

Table 4.5 summarizes the loading history and the measured displacements for Specimen CST3. Figures 4.25 and 4.26 show the hysteresis response and the loading histories for shear and axial loading (T/V ratio = 0.3), respectively. The measured strains in the steel anchor bars for shear and axial forces are presented in Figures 4.27 and 4.28, respectively. Figures 4.29(a) and 4.29(b) show photographs of the test specimen at yielding and failure, respectively. Figure 4.30 shows the distribution of strain along the length of the steel bar at the ultimate load. The following behavioural aspects were noted during testing and analysis of the test results:

Cycles 1-3 $V=40$ kN $T=15$ kN The load-deflection curves remained linear elastic and no cracks were noted.

Cycles 4-6 $V=80$ kN $T=25$ kN A splitting crack appeared along the tension leg, and a diagonal crack formed in the tension zone, which was mostly due to the shear force. The slope of load-deflection curves changed slightly (Figures 4.25 and 4.26). The area under the hysteresis loops was small indicating that negligible energy was dissipated. The horizontal and axial displacements were 1.6 mm and 1.3 mm, respectively (Table 4.5). During the last cycle (Cycle 6), the reinforcing bar yielded at Location 1 (Figures 4.27 and 4.28). The measured horizontal and axial displacements (Δ_y) were 1.8 mm and 1.5 mm, respectively. The maximum width of splitting and diagonal cracks were 0.25 mm and 0.3 mm, respectively. Photograph in Figure 4.29(a) shows the connection at yielding. Due to this yielding, the slope of load deflection curves reduced indicating decrease of connection stiffness (Figures 4.25 and 2.26). The area under the hysteresis loops increased during the positive loads (shear and axial tensile), which was mostly due to effect of the application of the axial load in terms of the cracking of concrete and yielding of the steel reinforcement.

Cycles 7-9 $V=110$ kN $T=35$ kN This stage was implemented using displacement control with a maximum deflection equal to $2\Delta_y$. The measured horizontal and axial displacements were 3.4 mm and 2.9 mm, respectively. A splitting crack appeared along the compression leg while the splitting crack in the tension zone extended along the entire length of the anchor bar, and other diagonal cracks formed. The maximum splitting and

diagonal crack widths during the last cycle were 1.0 mm and 1.5 mm, respectively.

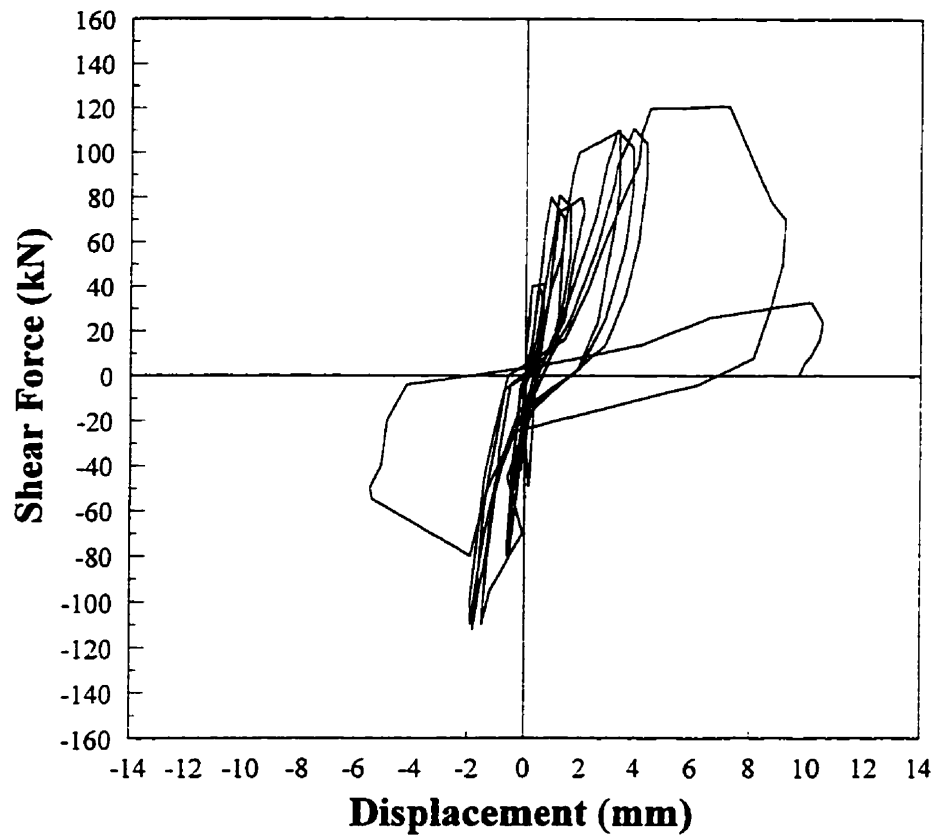
Cycle 10-11 $V=120$ kN $T=30$ kN This stage was involved using displacement control with a maximum deflection of $\Delta = 4\Delta_y$. As the shear and axial loads of 120 kN and 30 kN, respectively, were applied, the horizontal and axial displacements of 7.2 mm and 5.7 mm were measured. The cracks opened significantly and the displacement increased, while the shear force dropped to about 70 kN. The area under the hysteresis loops increased significantly. During the last cycle of this stage (Cycle 11), as the shear force of 33 kN and axial force of 10 kN were applied, the stiffness of the connection reduced very significantly and the connection underwent large displacements (Figures 4.25 and 4.26). The specimen could not resist any more load and the test was terminated. Since the hysteresis loop during last cycle was not stable, the displacement during the previous cycle (Cycle 10) was used to calculate the connection ductility. The maximum horizontal and axial displacements during last cycle were 8.7 mm and 8.5 mm, respectively. These gave a displacement ductility ratio of 4.

Since the strain gauges were lost during the casting of the specimen, except for Locations 1 and 4, there was no recorded data for these locations. Assuming linear strain distribution along the anchor leg at the ultimate load (Figure 4.30), about one-half of the length of tension reinforcing bar leg had yielded. The maximum shear and axial strengths of 120 kN and 35 kN, respectively, were achieved. The failure mode was due to bond failure along the tension leg (splitting), the crushing of the concrete between the legs of the anchor bar close to the edge, and the crushing-spalling of the concrete around the steel angle [Figure 4.30(b)]. The mode of failure was similar to that for the Specimen CS. The hysteresis response showed large loops in the positive direction after general yielding of the reinforcing anchor, which is an indicator of the connection energy dissipation. The experimental results showed that if less than 30% of the shear force is applied simultaneously and cyclically in the axial direction, the strength of the connection does not reduce significantly.

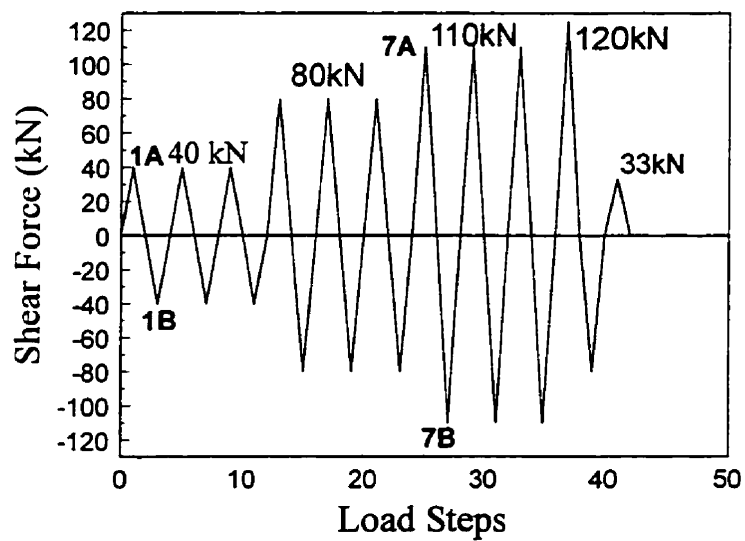
Table 4.5 Summary of Loading History for Specimen CST3

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.6 , -0.1	±40	+0.6 , +0.1	±15
Cracked	4A - 5B	+1.6 , -1.0	±80	+1.3 , +0.3	±25
±1 Δ y	6A - 6B	+1.8 , -1.0	±80	+1.5 , +0.3	±25
±2 Δ y	7A - 9B	+4.4 , -1.9	±110	+3.9 , +0.9	±35
±5 Δ y	10A - 10B	+8.7 , -5.5	±120	+8.3 , +0.9	±30
Failure	11A	+10.1	+33	+9.2	+10

* Maximum displacement at the peak load of the last cycle

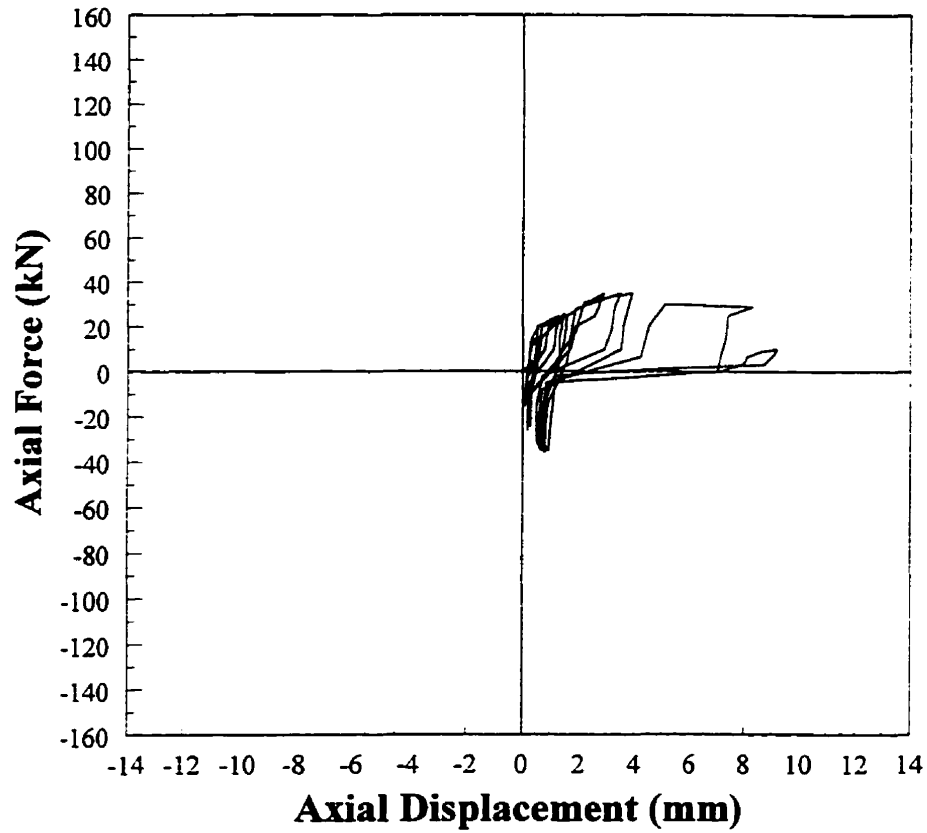


(a) Hysteresis response

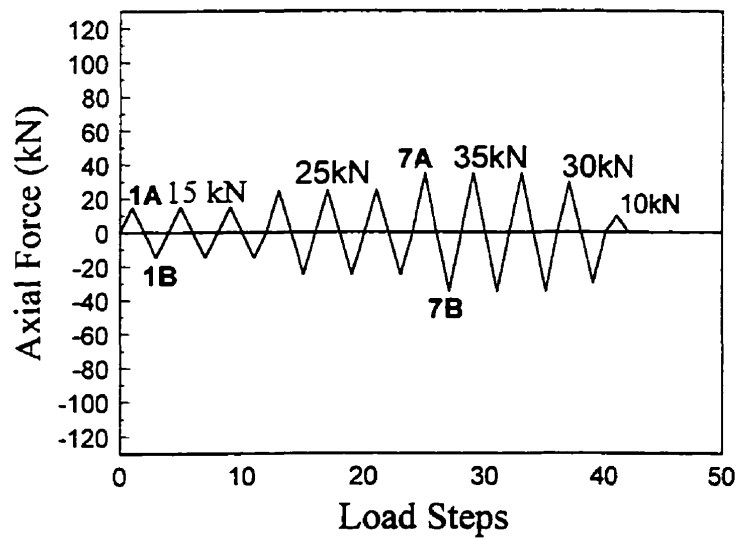


(b) Loading history

Figure 4.25 Hysteresis shear response and loading history for Specimen CST3

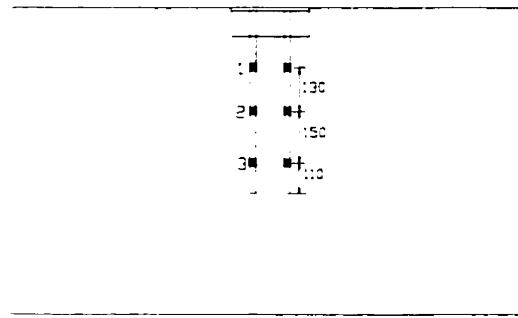


(a) Hysteresis response

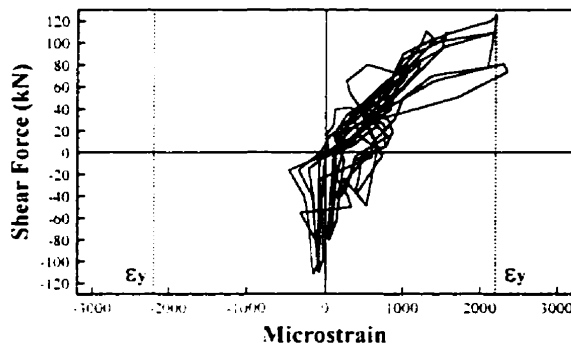


(b) Loading history

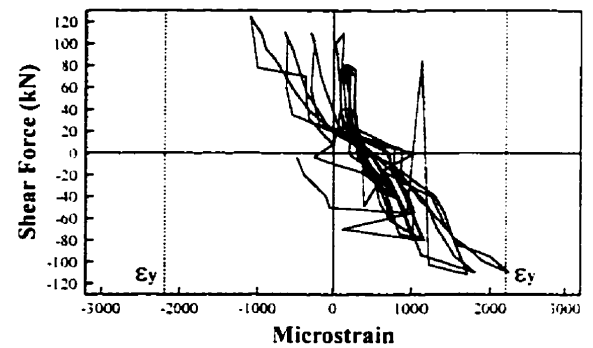
Figure 4.26 Hysteresis axial response and loading history for Specimen CST3



(a) Location of strain gauge

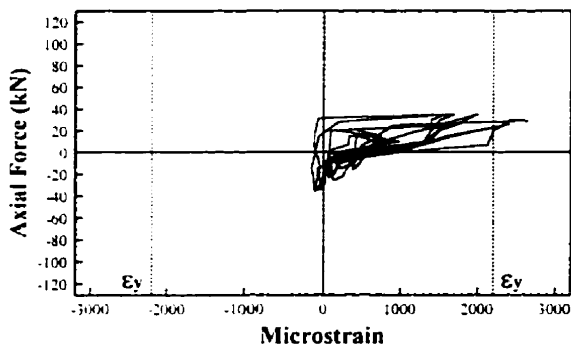


(b) Strains at Location 1, Shear response

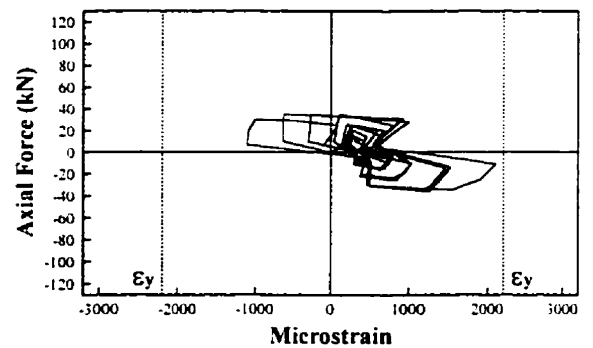


(c) Strains at Location 4, Shear response

Figure 4.27 Strains along length of anchor in Specimen CST3 (Shear response)

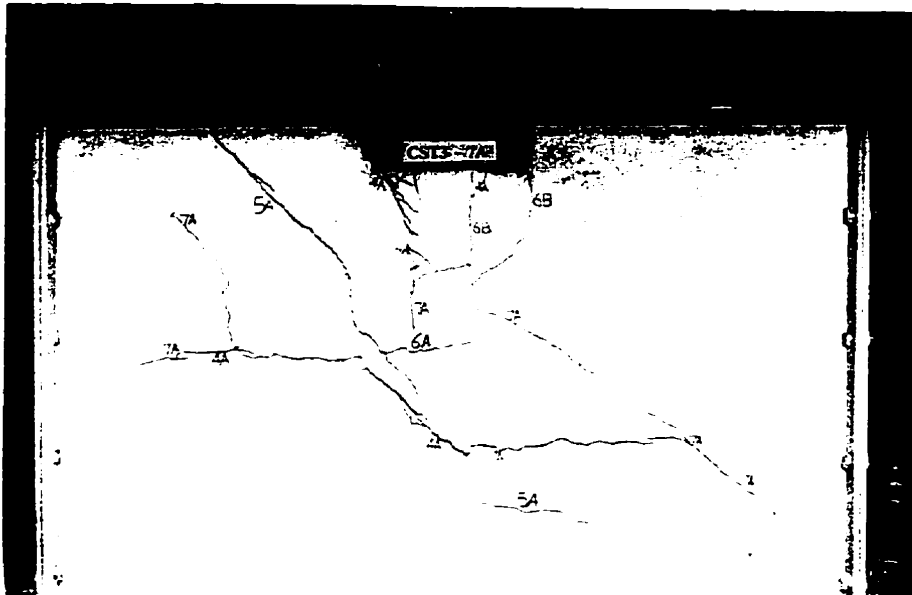


(b) Strains at Location 1, Axial response



(c) Strains at Location 4, Axial response

Figure 4.28 Strains along length of anchor in Specimen CST3 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}}=110 \text{ kN}$, $P_{\text{axial}}=35 \text{ kN}$



(b) Connection after failure, $P_{\text{ult, shear}}=120 \text{ kN}$, $P_{\text{ult, axial}}=30 \text{ kN}$

Figure 4.29 Photographs of Specimen CST3

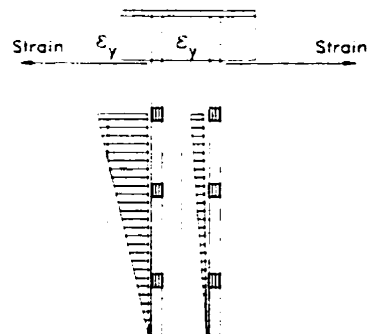


Figure 4.30 Strain distribution along the length of bar prior to failure of Specimen CST3

4.2.7 Interaction Diagram for Series C

In order to design safe precast structural systems, it is important to know the strength of the connections under the various combinations of forces induced during an earthquake. Therefore, a detailed investigation was carried out to develop the shear force-axial force interaction curve and the equation for this interaction curve.

The experimental behaviour and the ultimate capacities of the specimens in Series C, subjected to different combination of cyclic shear and axial loads, were reported in the previous sections (Sections 4.2.2 to 4.2.6). Based on the results obtained, and as summarized in Table 4.6, the maximum cyclic shear strengths, expressed as V/V_0 are plotted versus the cyclic axial strength ratios, T/T_0 , in Figure 4.31. The experimental interaction equation was fitted to the experimental results using the best-fit program, giving the following interaction equation:

$$\left(\frac{V}{V_0}\right) + \left(\frac{T}{T_0}\right)^2 \leq 1.0 \quad (1)$$

where, V is the applied shear force, V_0 is the maximum shear force that connection can resist in pure cyclic shear, T is the applied axial force, and T_0 is the axial maximum force that can be resisted by the connection. The material resistance factors must be included to adopt this equation for design purposes (see Chapter 7).

Figure 4.31 shows that the strength of the connection under the cyclic shear force decreases when cyclic axial force is applied simultaneously. The reduction in the strength of the connection is due to the fact that the cyclic axial forces induced additional stresses at the interface of the concrete and the reinforcing bar anchors (bond stress). Furthermore, it stresses the concrete surrounding the connection steel angle, resulting in deduction of the bond strength when the shear forces are applied. It is noted that if less than 30% of the shear force is applied simultaneously and cyclically in the axial direction, the strength of connection does not reduce significantly. Moreover, the cracking load and the slope of the load-deflection response for shear loading (shear stiffness) consequential decrease due to the application of the axial forces. Similarly, cyclic axial strength decreases when an cyclic shear force is applied simultaneously.

Table 4.6 Summary of interaction test results for Series C

Test type	Ratio of axial to shear (T/V)	Ultimate strength (kN)		Ratios	
		Shear (kN)	Axial (kN)	V/V ₀	T/T ₀
CS	-	125	-	1.00	0.0
CST1	0.7	100	65	0.80	0.45
CST2	1.0	90	85	0.72	0.57
CST3	0.3	120	30	0.96	0.20
CT	-	-	150	0.0	1.0

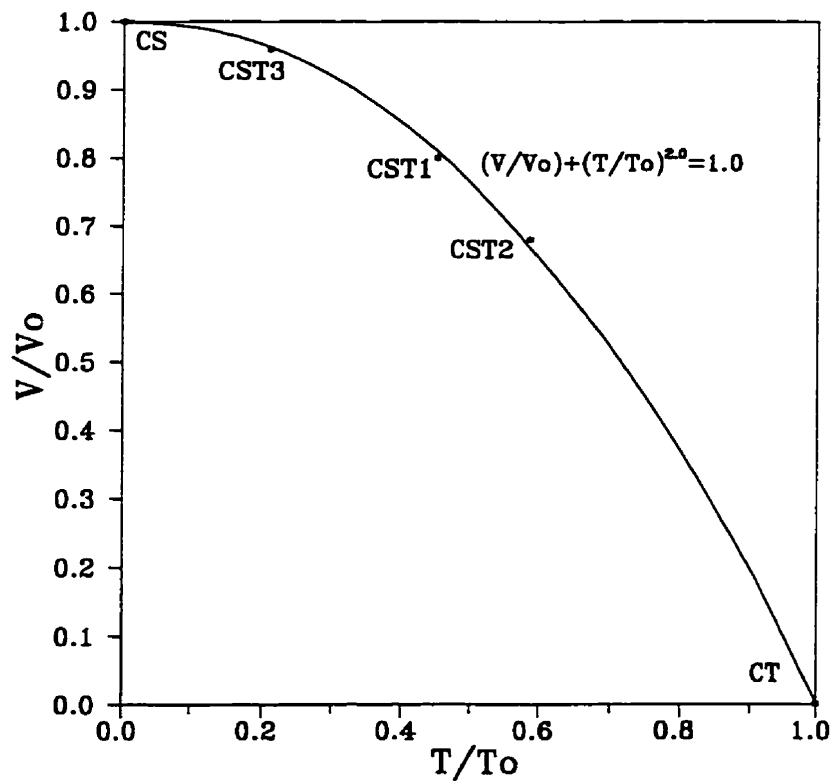


Figure 4.31 Normalized interaction curve for specimens in Series C

4.3 Test Series D: Connection with three deformed reinforcing bars welded to steel angle

This connection consisted of 3 No 15 deformed bar, oriented at 45°, 90° and 135°, and welded to a 10 mm thick steel angle (see Figure 2.15). A complete description of the connection can be found in Section 2.4. The design and determination of the ultimate capacity was performed following the same technique as for the Test Series A. Six identical specimens were constructed and tested subjected to different load combinations—one under monotonically increasing shear load (Specimen DMS), and the remaining specimens (Specimens DS, DT, DST1, DST2 and DST3) subjected to combined cyclic shear and axial loads with the various values of the T/V ratio.

4.3.1 Specimen DMS

Figure 4.32 shows the shear force-horizontal displacement curve for the specimen. The load versus measured strains along the length of bar are presented in Figure 4.33. The profile of the strains prior to failure versus the distance along the length of bar is shown in Figure 4.34. Figure 4.35 shows photographs of the connection at the yield stage and after failure. The following general behavioural aspects were noted during the testing and analysis of the experimental results.

Stage 1 70 kN The load-deflection remained linear (Figure 4.32) before the first crack appeared. The maximum crack width was 0.3 mm.

Stage 2 95 kN The splitting crack extended along the bar, while a diagonal crack appeared perpendicular to this splitting crack. The maximum strain was 1966×10^{-6} which is close to the yield strain (Figure 4.33).

Stage 3 145 kN A crack appeared across the concrete panel in the tension zone. The measured displacement was 0.98 mm and the maximum crack width was 0.3 mm. There was a significant increase in the displacement for a small increment in the applied load.

Stage 4 205 kN The reinforcing bar yielded with the horizontal displacement at yield being $\Delta_y = 2.53$ mm, and the measured crack width was 1.0 mm [Figure 4.35(a)]. The slope of the load-deflection curve kept on decreasing up to the ultimate load of 226 kN. The connection failed due to disintegration of the concrete along the diagonal crack [Figure 4.35(b)], with the maximum displacement being 7.7 mm given a displacement

ductility ratio of 3.1.

The load-displacement response showed considerable flexible behaviour after the yielding of the reinforcing bar in the tension zone. It was noted that the cracking of the concrete at the yield load level was significant. The mode of the failure was due to complete splitting of the concrete along the diagonal crack intersecting the tension reinforcing bar and extending across the middle of concrete panel as shown in Figure 4.35(b). The variation of the strains along the length of the reinforcing bar at the ultimate load (Figure 4.34) indicates that almost the entire reinforcing bar yielded in tension. No yielding was noted in the compression leg.

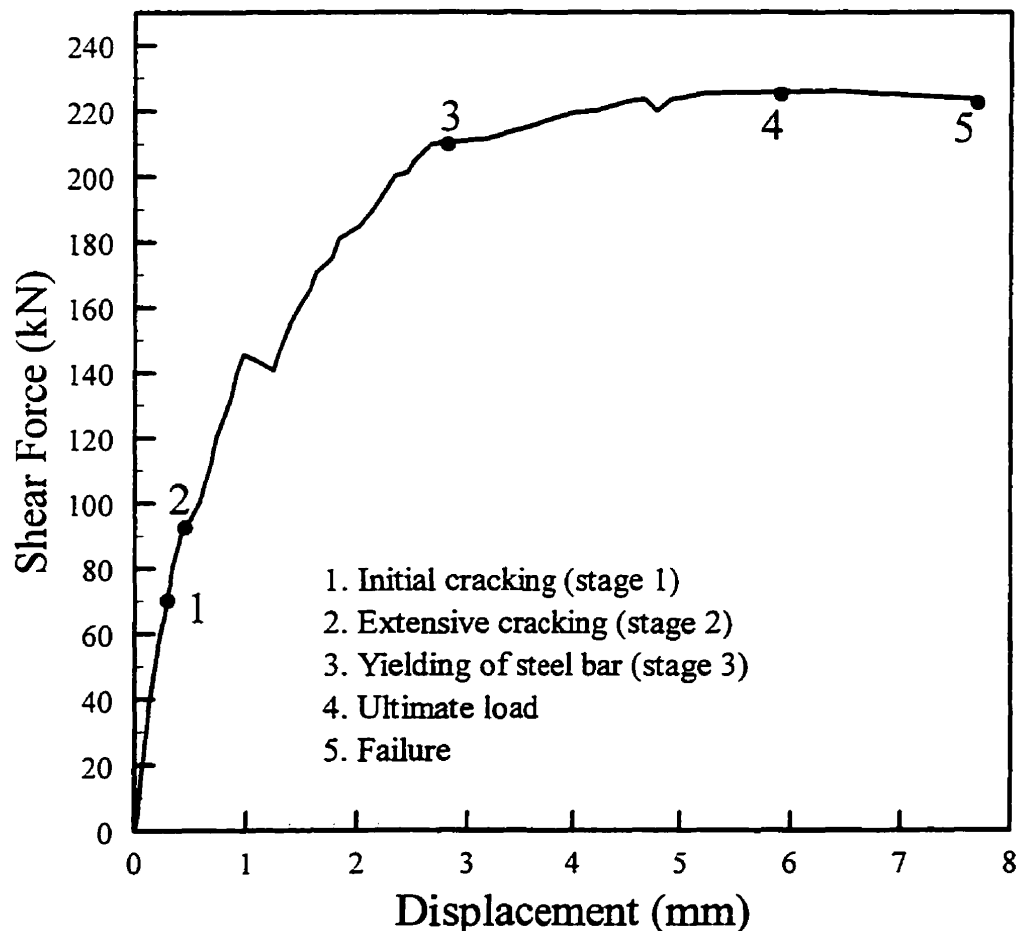
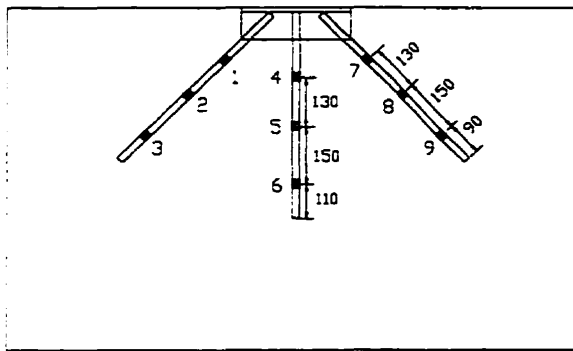
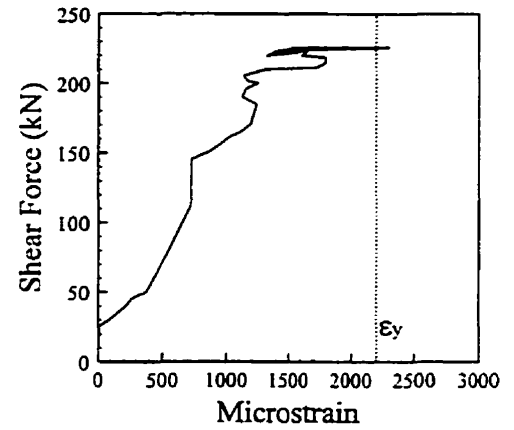


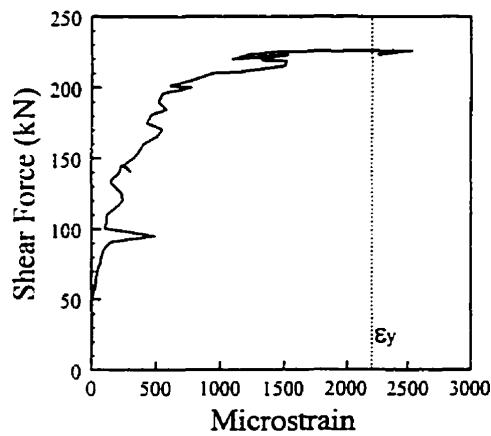
Figure 4.32 Shear force vs. horizontal displacement curve of Specimen DMS



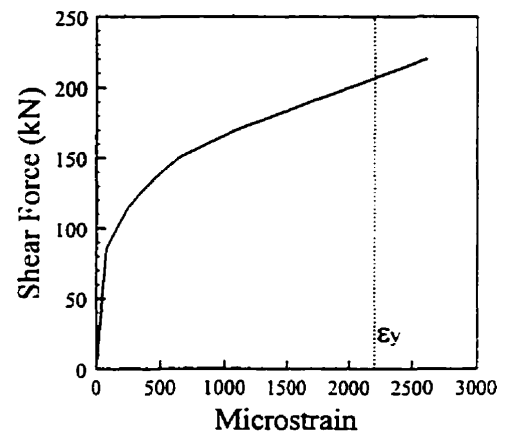
(a) Location of strain gauges



(b) Strains at Location 1



(c) Strains at Location 2



(c) Strains at Location 3

Figure 4.33 Measured strains along the length of anchor for Specimen DMS

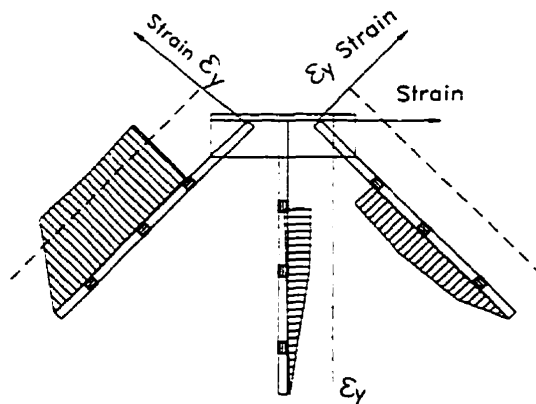
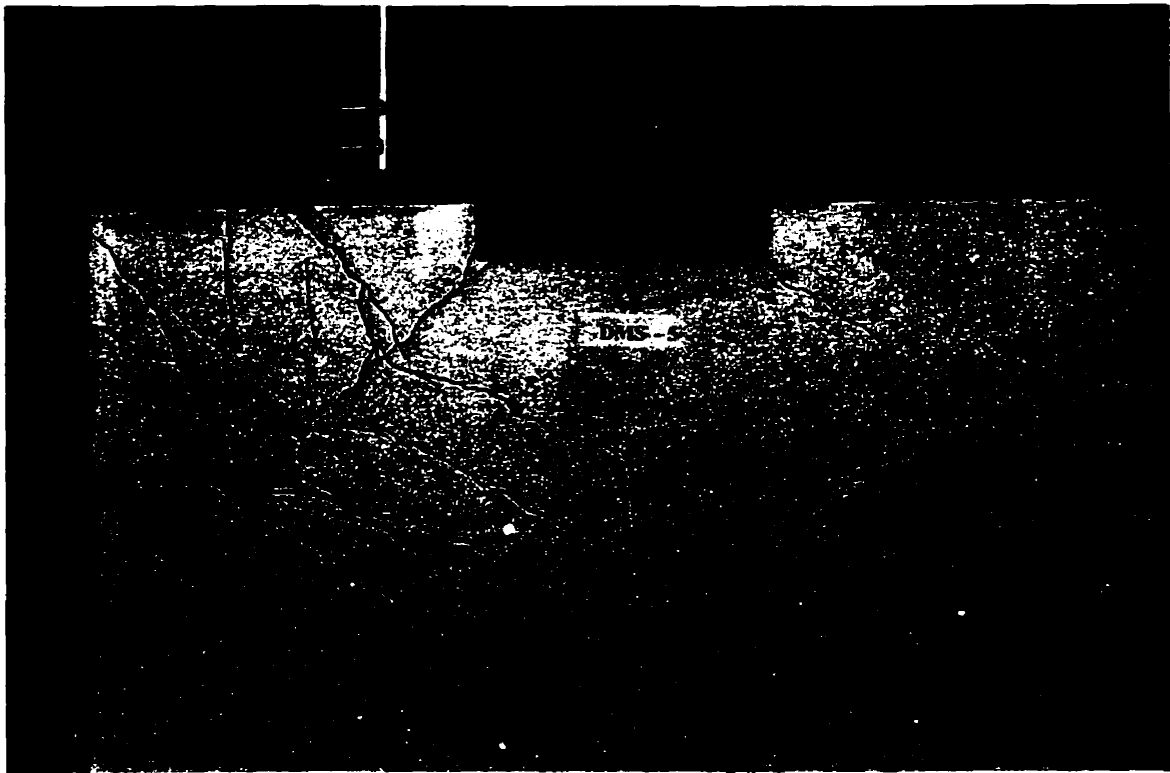
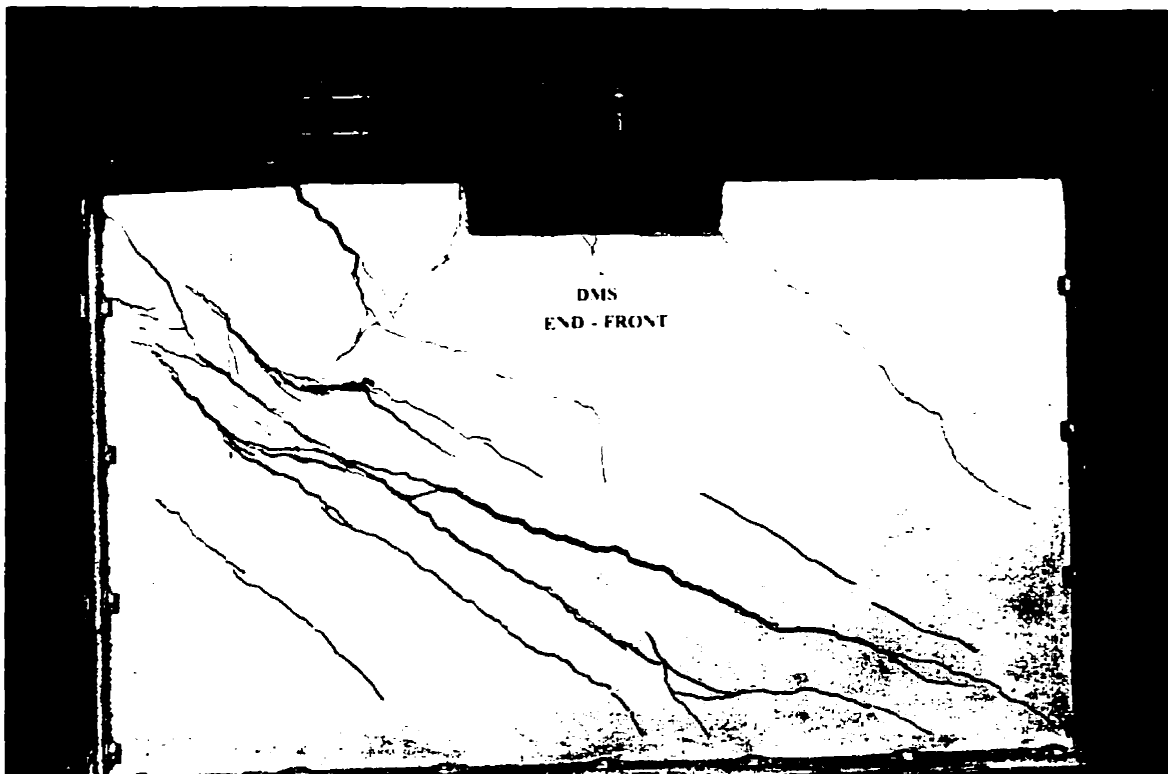


Figure 4.34 Strain distribution along the length of bar prior to failure of Specimen DMS



(a) Connection at first yield, Δ_v , $P=205$ kN (Stage 4)



(b) Connection after failure, $P_{ult} = 226$ kN

Figure 4.35 Photographs of Specimen DMS

4.3.2 Specimen DS

The Specimen DS from the Series D was loaded under reversed cyclic shear only, as summarized in Table 4.7. Figure 4.36 shows the loading history and the hysteresis load-displacement response of the specimen. The variation of the strains with applied shear force along the length of the anchor is presented in Figure 4.37. The distribution of the strains with the distance along the length of reinforcing bar prior to failure is shown in Figure 4.38. The photographs of the specimen at the yield level and after failure are shown in Figures 4.39(a) and 4.39(b), respectively. The following observations of the various behavioural aspects were made during the testing and from the experimental analysis of the results.

Cycles 1-3, 65 kN There were no cracks during Cycles 1 and 2. The first crack appeared along tensile leg during Cycle 3A, however, the load-deflection response remained linear up to the end of this load stage.

Cycles 4-6, 120 kN The previous splitting crack extended along the legs and a diagonal cracks formed perpendicular to this crack. This caused a change in the slope of the load-displacement response (Figure 4.36). The maximum diagonal crack width at the end of the bar anchor was 0.4 mm. These cracks propagated during the following cycles with nearly symmetrical crack patterns. Nonlinear response started immediately after cracking (see Figure 4.36). No noticeable energy was dissipated up to the end of this stage.

Cycles 7-9, 150 kN The anchor bar yielded at Location 1 (Figure 4.37) resulting in increase in the displacement to $\Delta_y=1.3$ mm. The diagonal crack widened considerably to a width of about 0.8 mm. A similar crack pattern was observed during the negative loading cycle (see Figure 4.39(a)). The area under the loops was not significant, showing an insignificant energy dissipation.

Cycles 10-12, 180 kN This stage was implemented using displacement control with the maximum displacement being, $2\Delta_y=2.6$ mm. The anchor bar yielded at Location 3 (Figure 4.37) indicating that yielding had developed along the bar length. It was noted that the reinforcing bar had yielded at Location 2 during the last cycle of previous load stage (Cycle 8-9). The strain gauge at Location 2 had been lost during casting. The cracks propagated all over the specimen, and were symmetric due to application of

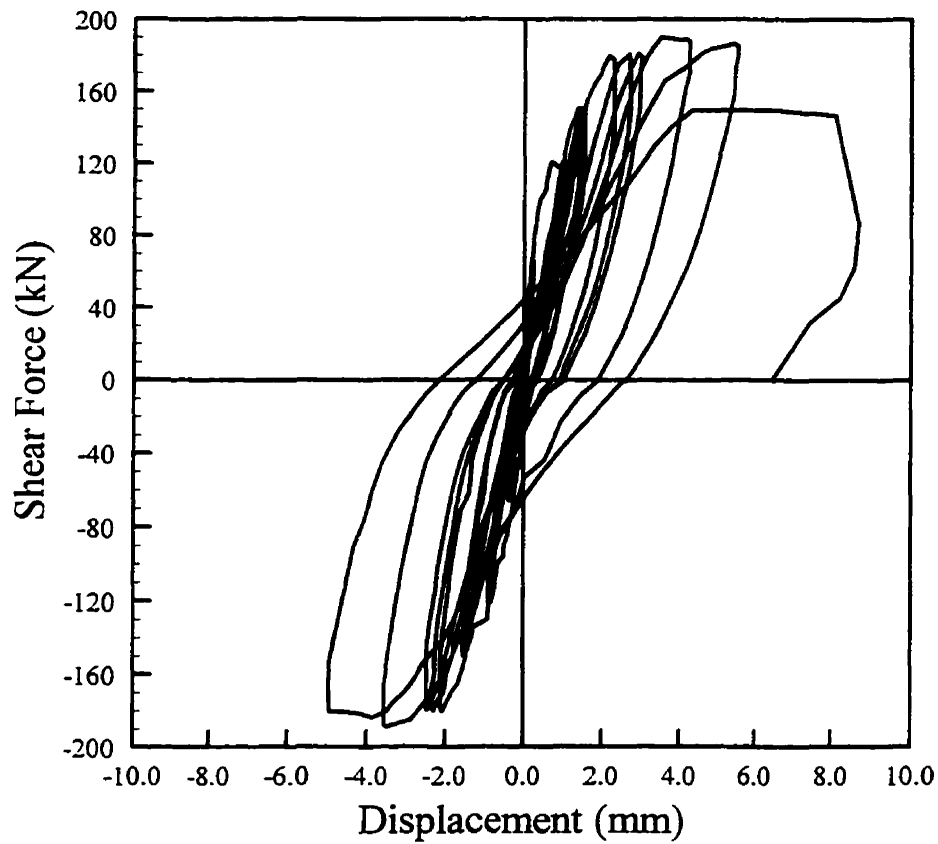
reversed loading cycles. Due to the development of yielding and crack propagation, the area under the loops increased, with more energy being dissipated by the connection. The maximum crack width at peak of last cycle was 1.9 mm.

Cycles 13-15, 190 kN This stage was controlled by a maximum displacement equal to $3\Delta_y$. No more cracks appeared, however, the cracks widened and the stiffness of connection (slope of the load-deflection curve) kept on decreasing. The area under the loops increased significantly due to increasing development of yielding along the bar length. This resulted in significant energy dissipation by the connection. During Cycle 15A, the maximum load was 150 kN when the connection underwent a large displacement, and the load dropped to 100 kN. The connection could not carry any more load when the test was stopped. The ultimate displacement prior to failure was 8.1 mm giving a displacement ductility ratio of 6.2. The profile of the strains along the anchor bar legs at the ultimate load shows that almost the entire length of the tension leg of the anchor had yielded (Figure 4.38). The final failure mode was due to the diagonal concrete cracking at the end of the tensile leg which extended to the compression zone [see Figure 3.39(b)]. The hysteresis response shows large loops beyond the yielding of the steel bar, indicating the energy dissipation capability of the connection after yielding of anchor bar.

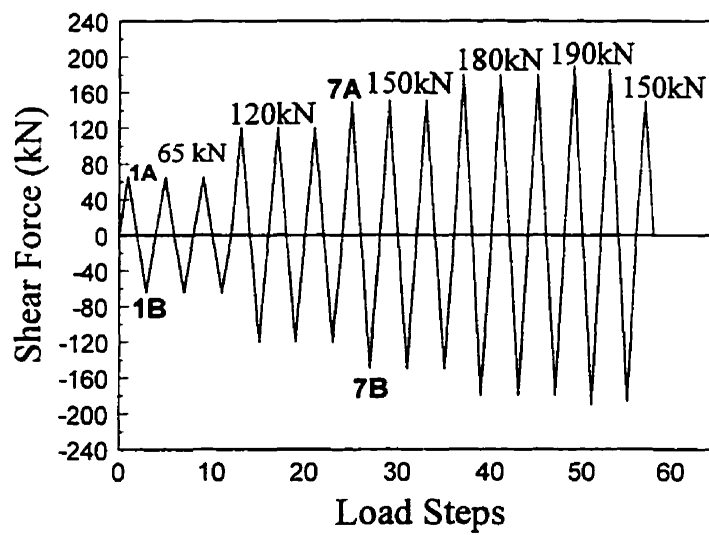
Table 4.7 Summary of Loading History for Specimen DS

Cycle	Load Stages	Horizontal Displacement (mm)*	Shear Force (kN)
Elastic	1A - 2B	+0.2 , -0.4	±65
Cracked	3A - 3B	+0.3 , -0.5	±65
Extensive cracking	4A - 6B	+1.0 , -0.9	±120
±1Δ _y	7A - 9B	+1.5 , -1.5	±150
±2Δ _y	10A - 12B	+3.0 , -2.5	±180
±3Δ _y	13A - 14B	+5.5 , -4.9	±190
Failure, +6.2Δ _y	15A	+8.1	+150

* Maximum displacement at the peak load of the last cycle

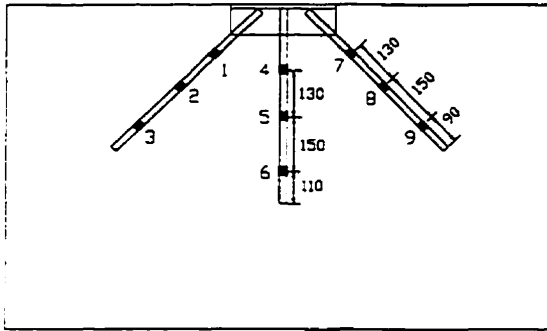


(a) Hysteresis response

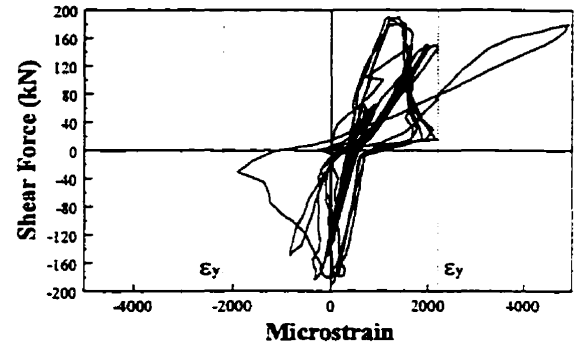


(b) Loading history

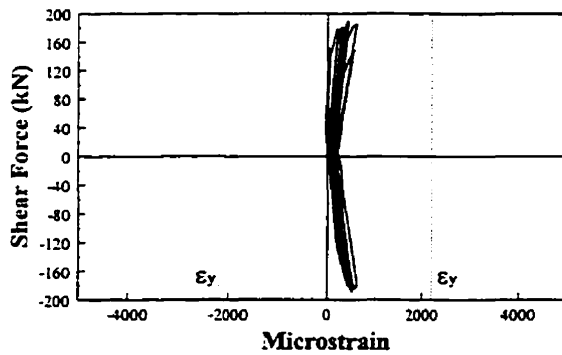
Figure 4.36 Hysteresis shear response and loading history for Specimen DS



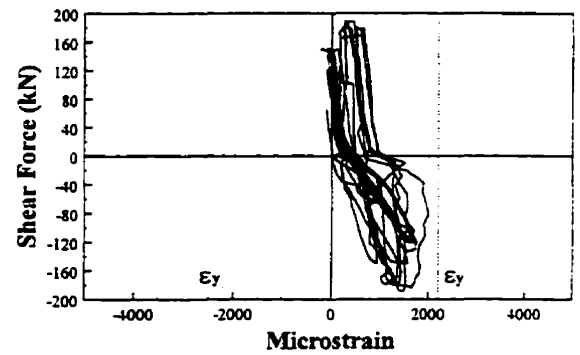
(a) Location of strain gauges



(b) Strains at Location 1



(c) Strains at Location 4



(d) Strains at Location 7

Figure 4.37 Measured strains along the length of the anchor for Specimen DS

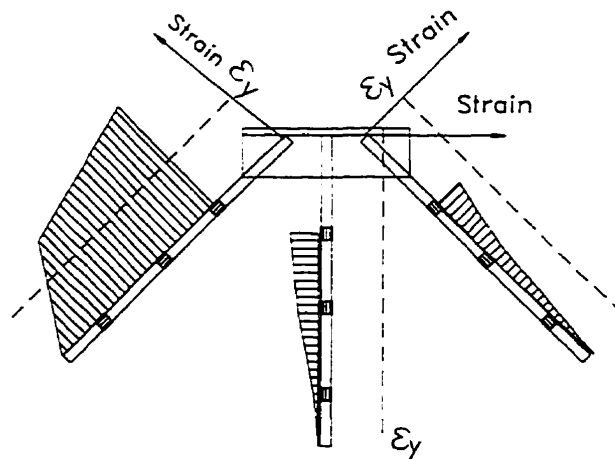
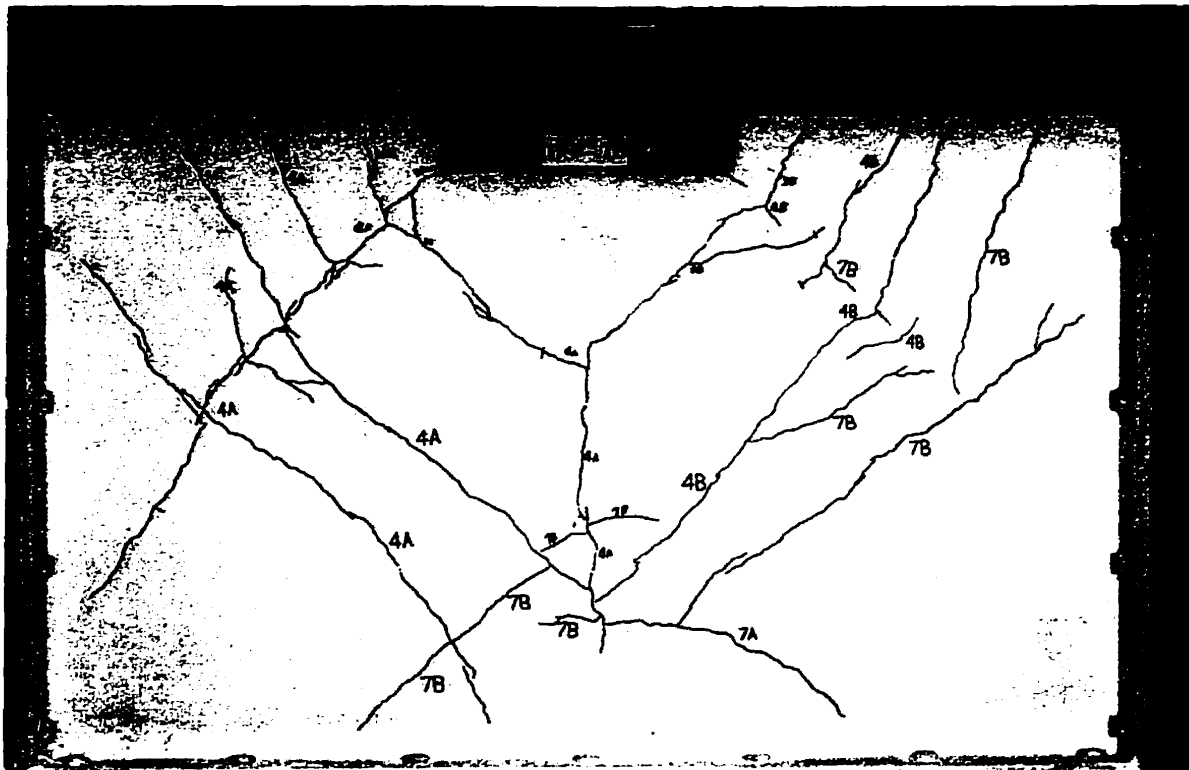
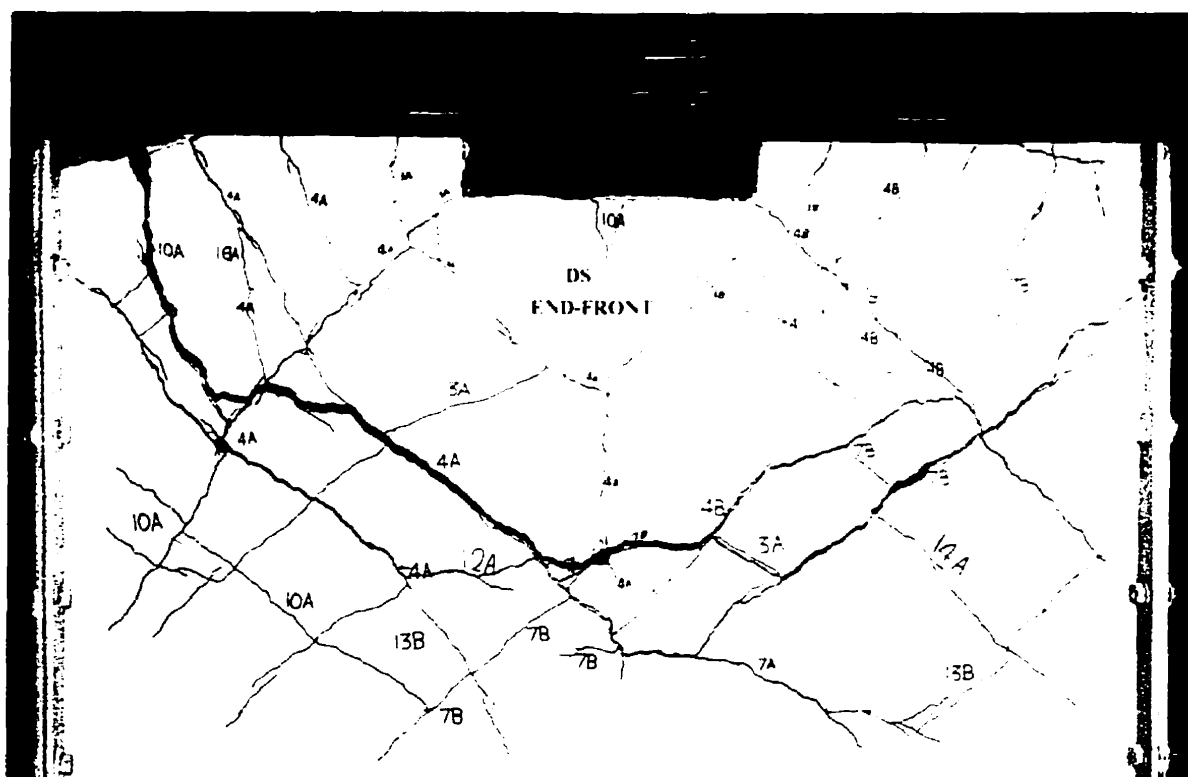


Figure 4.38 Strain distribution along the length of bar prior to failure of Specimen DS



(a) Connection at first yield, Δ_y , $P=150$ kN



(b) Connection after failure, $P_{ult} = 150$ kN

Figure 4.39 Photographs of Specimen DS

4.3.3 Specimen DT

The loading history and the associated displacements are given in Table 4.8. Figure 4.40 shows the hysteresis response and the loading history of the specimen. The applied axial load versus the strains at different locations along the reinforcing bar are shown in Figure 4.41. Figure 4.42 illustrates the distribution of the axial strain at the ultimate load versus the distance along the straight part of the anchor bar. The photographs of the connection at the yield stage and after failure are presented in Figures 4.43(a) and 4.43(b), respectively. The following behavioural aspects were noted based on the observations during testing and analysis of the experimental results.

Cycles 1-3, 50 kN There was no cracking, and the response was linear-elastic.

Cycles 4-6, 65 kN Two diagonal cracks appeared at about the mid-length and at the end of each inclined leg, and one splitting crack was noted along the vertical leg. This caused a change in the slope of the load-deflection curve and the commencement of nonlinear behaviour. The maximum diagonal crack width was 0.1 mm at the end of the bar. The displacement was small (0.3 mm) and the area under the hysteresis loops were very small (Figure 4.40).

Cycles 7-9, 105 kN Diagonal and splitting cracks widened and extended, resulting in sharp change in the stiffness of the connection (Figure 4.40). The crack pattern was symmetrical with a maximum crack width of 0.2 mm. The measured displacement (0.6 mm) was about three times the elastic one (0.2 mm).

Cycles 10-12, 135 kN The reinforcing bar anchor yielded at a load of 130 kN (Figure 4.41) resulting in a significant increase in the displacement and a decrease in the stiffness of the connection (Figure 4.40). The displacement at yield (Δ_y) was =1.3 mm. The cracks extended toward the centre of the test specimen and the concrete at the corner of the angle started to separate [Figure 4.43(a)]. During the following cycles (Cycles 11 and 12) the area under the hysteresis loops did not increased significantly.

Cycles 13-15, 155 kN This stage was implemented using displacement control with a maximum displacement equal to $2\Delta_y$. The cracks propagated all over the specimen, and the diagonal crack at the end of the anchor bar widened with a measured crack width equal to 1.2 mm. The stiffness of the connection kept on decreasing and the area under

the loops increased, resulting in an increase in the energy dissipation because of extensive cracking and development of yielding through the steel anchor bars.

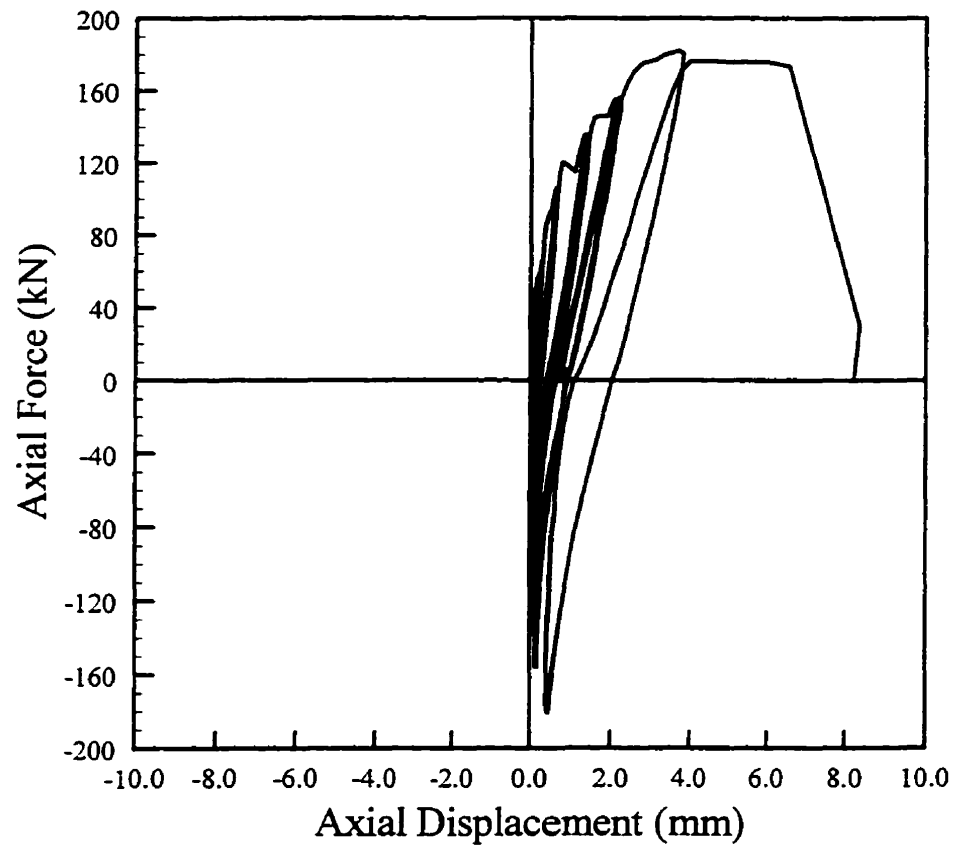
Cycles 16-17, 180 kN This stage was characterized by a maximum displacement of $3\Delta y$. The ultimate strength of 180 kN was achieved during Cycle 16. The applied load decreased to 175 kN during Cycle 17 when the specimen failed. This is displayed in the hysteresis response with a large plateau. Then the load dropped to about 38 kN and the test was stopped to avoid damage to the instrumentation. The ultimate displacement prior to failure was 6.5 giving a displacement ductility of ratio 5.0.

The axial strain variation along the bar anchor at the ultimate load (Figure 4.42) indicates that almost three-fourths of the length of the inclined legs had yielded. The mode of failure was due to continuous diagonal cracking at the end of all three anchor bars [see Figure 4.43(b)]. The hysteresis response shows large loops in the positive direction which was due to the development of yielding along the reinforcing bar and with extensive diagonal cracking. The small loops in the negative direction were due to the imperfect closing of cracks and small deformation of the concrete subjected to compressive stresses.

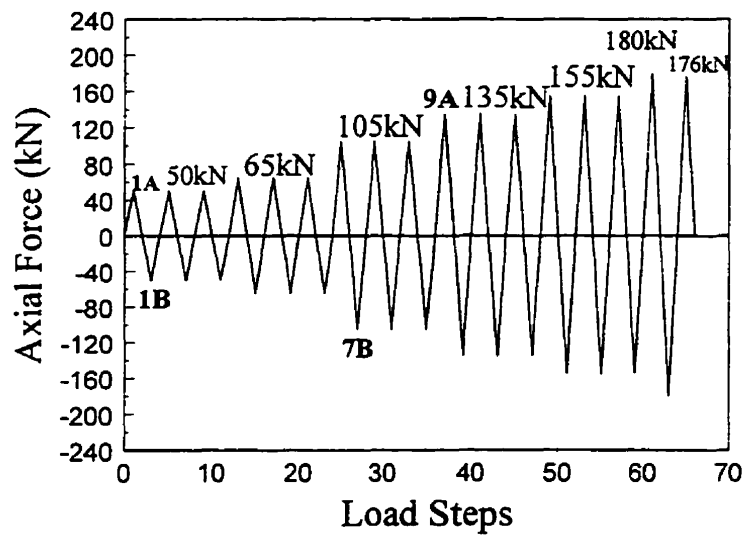
Table 3.8 Summary of Loading History for Specimen DT

Cycle	Load Stages	Axial Displacement (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.2 , +0.01	±50
Cracked	4A - 6B	+0.3 , +0.02	±65
Extensive cracking	7A - 9B	+0.6 , +0.03	±105
±1Δy	10A - 12B	+1.4 , +0.07	±135
±2Δy	13A - 15B	+2.6 , +0.2	±155
±3Δy	16A - 16B	+3.8 , +0.4	±180
Failure, 5Δy	17A	+6.5	+176

* Maximum displacement at the peak load of the last cycle

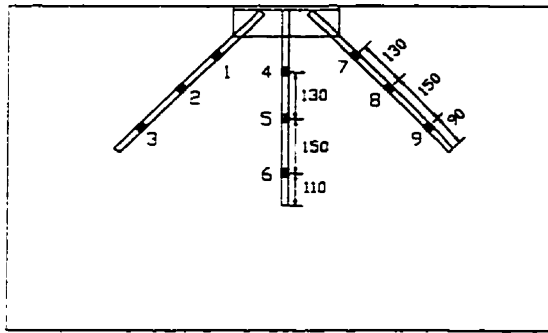


(a) Hysteresis response

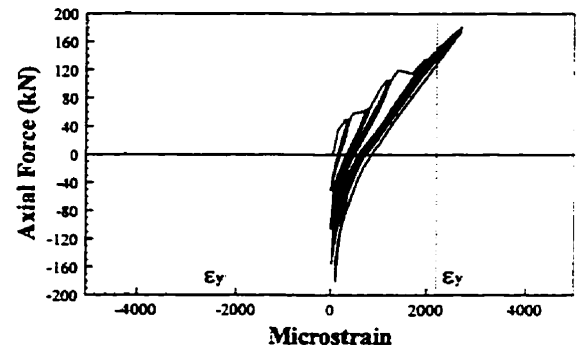


(b) Loading history

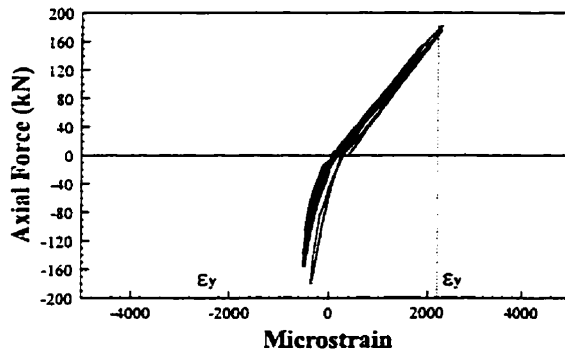
Figure 4.40 Hysteresis axial response and loading history for Specimen DT



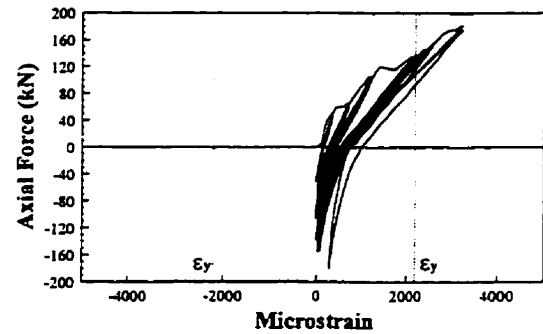
(a) Location of strain gauges



(b) Strains at Location 1



(c) Strains at Location 4



(d) Strains at Location 7

Figure 4.41 Measured strains along the length of the anchor for Specimen DT

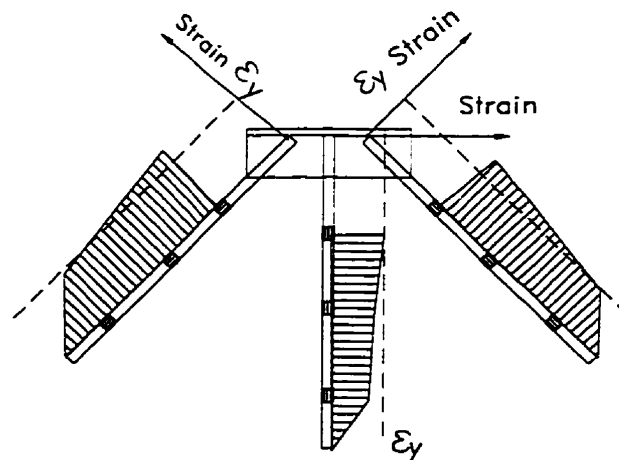
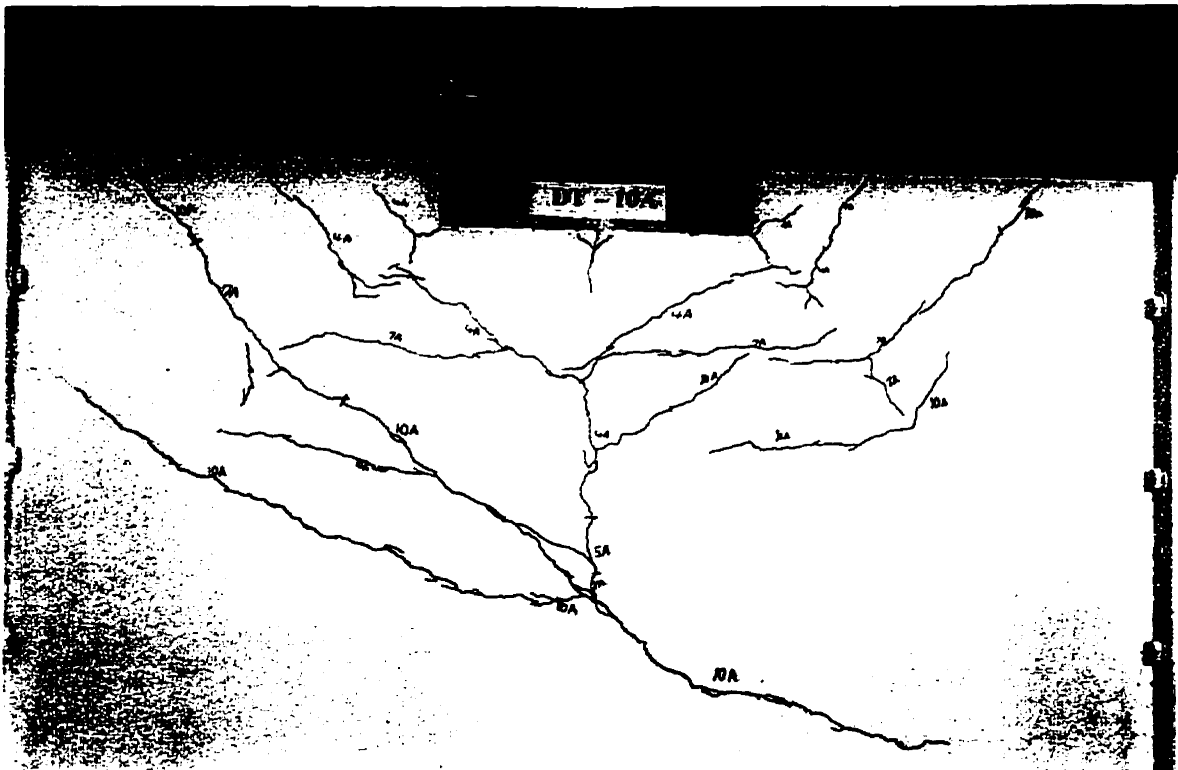
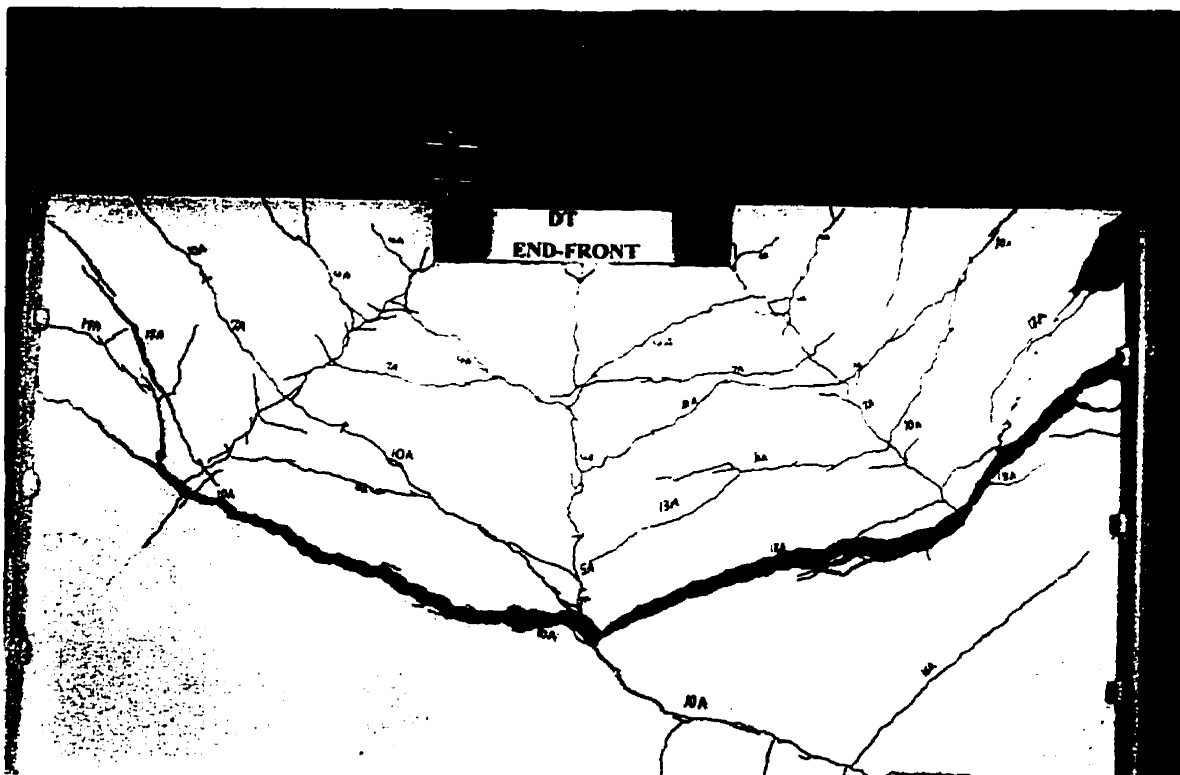


Figure 4.42 Strain distribution along the length of bar prior to failure of Specimen DT



(a) Connection at first yield, Δ_y , $P=135$ kN



(b) Connection after failure, $P_{ult} = 176$ kN

Figure 4.43 Photographs of Specimen DT

4.3.4 Specimen DST1

The loading history and the associated displacements are given in Table 4.9. Figures 4.44 and 4.45 show the hysteresis curves and the loading histories for the shear and the axial responses (T/V ratio = 0.7), respectively. The measured strains along the length of reinforcing bar versus the shear and the axial loads are presented in Figures 4.46 and 4.47, respectively. Figures 4.48(a) and 4.48(b) show the photographs of the specimen at the yielding stage and after failure, respectively. The distribution of strains with the distance along the length of anchor bar at the ultimate load is shown in Figure 4.49. The following notes on the various behavioural aspects were made from the observation during the test and from the analysis of the experimental results:

Cycles 1-3, $V=35\text{ kN}$ $T=25\text{ kN}$ There was no cracking and the response was basically linear elastic.

Cycles 4-6, $V=65\text{ kN}$ $T=45\text{ kN}$ A splitting crack appeared along the tensile leg and two diagonal cracks formed perpendicular to this splitting crack in the tension zone. The nonlinearity started for the shear response but it was not noticeable for the axial response (see Figures 4.44 and 4.49). The maximum diagonal crack width was 0.1 mm and the displacements were small.

Cycles 7-9, $V=95\text{ kN}$ $T=68\text{ kN}$ The reinforcing bar yielded at Location 1 (Figures 4.46 and 4.47). The horizontal and axial displacements (Δ_y) were 1.3 mm and 0.8 mm, respectively. As the yielding occurred, several diagonal and splitting cracks in tension zone and one major splitting crack along the compression leg were observed [see photograph in Figures 4.49(a)], resulting in reduction of the connection stiffness (Figures 3.44 and 3.45). The maximum splitting and diagonal crack widths were 0.33 mm and 0.5 mm , respectively. For both shear and axial responses, the area under the loops increased during the positive load (shear and axial tension), while in the negative (shear and axial compression) direction, this area was very small. The dissipated energy was not noticeable because of narrow loops during the negative loading cycle.

Cycles 10-12, $V=115\text{ kN}$ $T=80\text{ kN}$ This stage was implemented using displacement control with a maximum displacement equal to $2\Delta_y$. The horizontal and axial displacements were 2.5 mm and 1.6 mm, respectively. The steel anchor bar yielded at

Location 2, showing the extension of yielding along the tension leg. The load was maintained at the same level for the following cycles. The displacement increased which indicates a reduction of the connection stiffness and strength due to the cyclic loading. The area under the hysteresis loops increased, especially in the positive direction showing the energy dissipation capability of the connection throughout the development of yielding along the reinforcing bar. The maximum splitting and diagonal crack widths were 0.6 mm and 2.1 mm, respectively, at the end of this loading stage..

Cycles 13-14, $V=130\text{ kN}$ $T=85\text{ kN}$ This stage involved using the maximum displacement equal to $3.5\Delta_y$. The horizontal and vertical displacements were 4.4 mm and 3.2 mm, respectively. The area under the loops increased significantly and the maximum splitting and diagonal crack width were 0.8 mm and 5.0 mm, respectively.

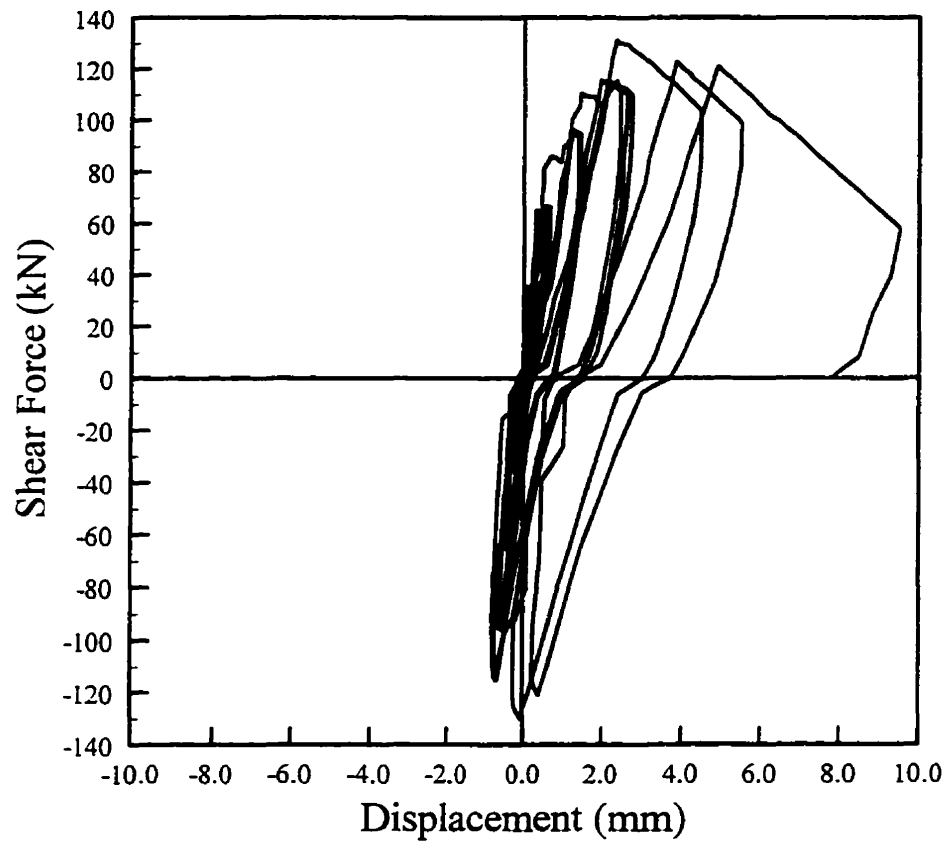
Cycle 15, $V=120\text{ kN}$ $T=75\text{ kN}$ The specimen underwent large displacements and it finally failed. The ultimate horizontal and axial displacements prior to failure were 5.1 mm and 8.2 mm, respectively, giving a displacement ductility ratio of 6.5.

The distribution of strains along the anchor bar legs at the ultimate load show that about one-half of the tension leg length had yielded (Figure 4.48). The failure mode was due to a large diagonal crack at the end of tension leg and splitting bond failure along the two inclined legs, see Figure 4.49(b). The results show that under combined cyclic axial and shear loading ($T/V=0.7$), the strength of the connection decreased to about 68 % and 47% of those for pure cyclic shear and pure cyclic axial strengths, respectively. The hysteresis response shows larger loops in the positive direction (combination of shear and tensile axial load) which was due to the extensive opening of the diagonal cracks and the development of yielding along the reinforcing bar. Smaller loops in the negative direction (shear and compression axial loads) was due to compressive axial load, imperfect closing of the cracks and lower deformability of concrete when subjected to compressive stresses. The area under the loops increased significantly after general yielding of the anchor bar resulting in a large energy dissipation.

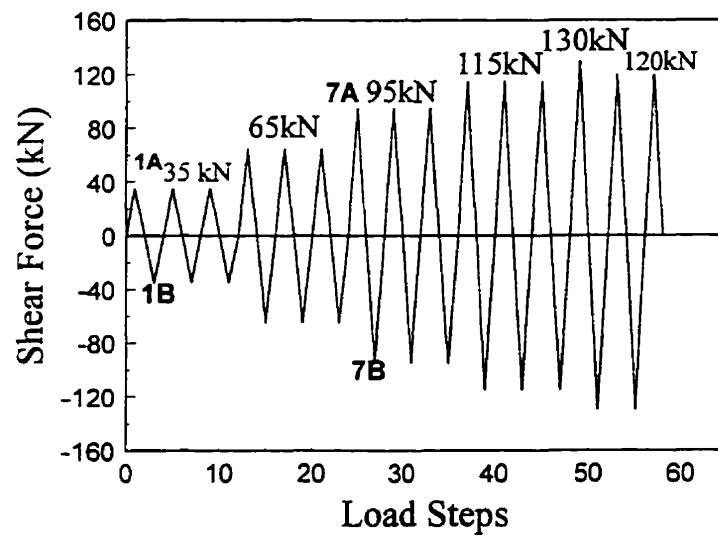
Table 4.9 Summary of loading history for Specimen DST1

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.2 , -0.2	±35	+0.3 , +0.02	±25
Cracked	4A - 6B	+0.6 , -0.4	±65	+0.5 , +0.04	±45
±1 Δy	7A - 9B	+1.5 , -0.6	±95	+1.2 , +0.1	±68
±2 Δy	10A - 12B	+2.7 , -0.8	±115	+2.1 , +0.6	±80
±3.5 Δy	13A - 14B	+5.1 , -0.3	±130	+5.5 , +2.5	±85
6.5 Δy , Failure	15A	+5.2	+120	+8.2	+75

* Maximum displacement at the peak load of the last cycle

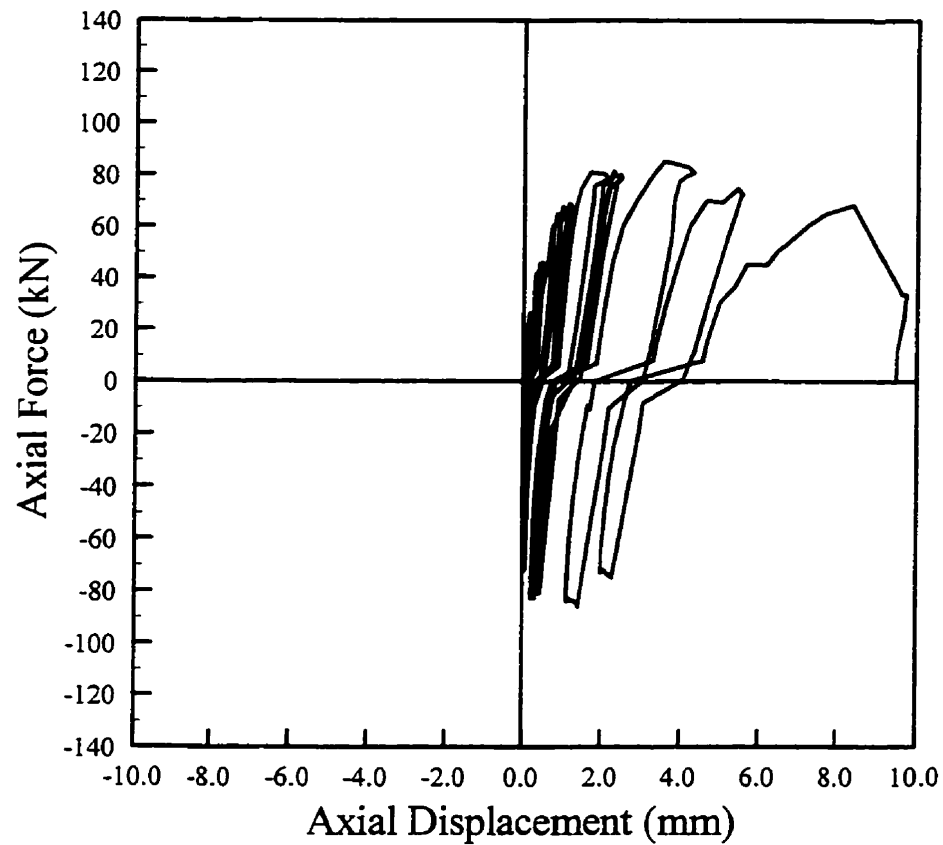


(a) Hysteresis response

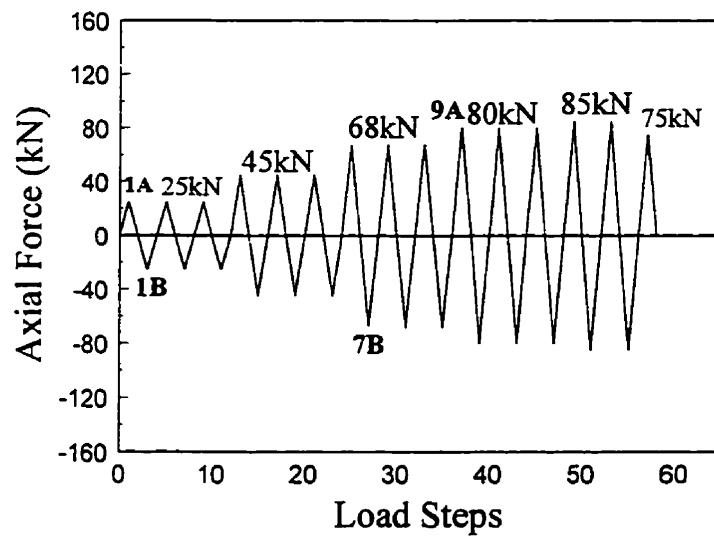


(b) Loading history

Figure 4.44 Hysteresis shear response and loading history for Specimen DST1

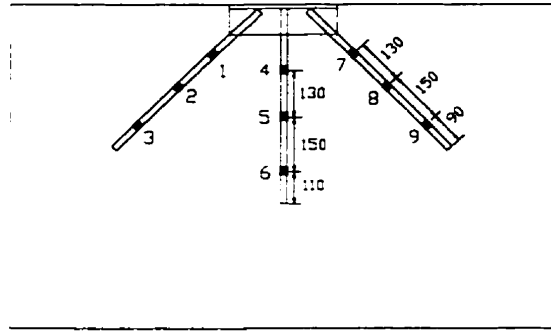


(a) Hysteresis response

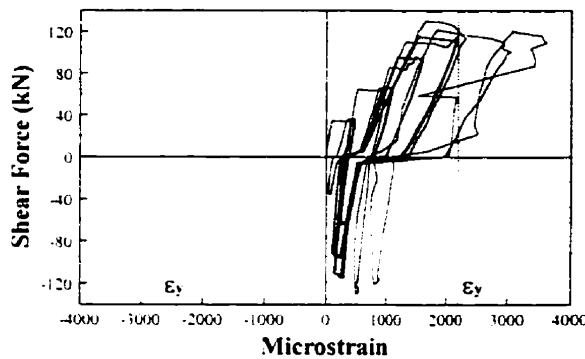


(b) Loading history

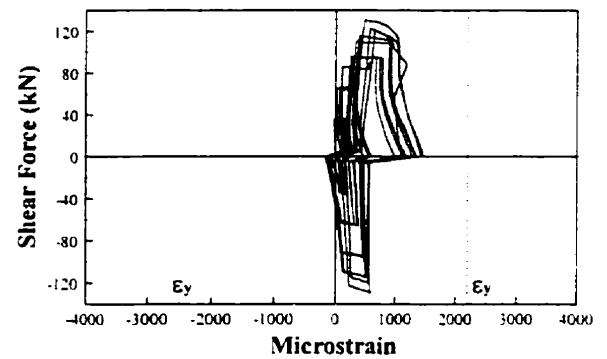
Figure 4.45 Hysteresis axial response and loading history for Specimen DST1



(a) Location of strain gauges

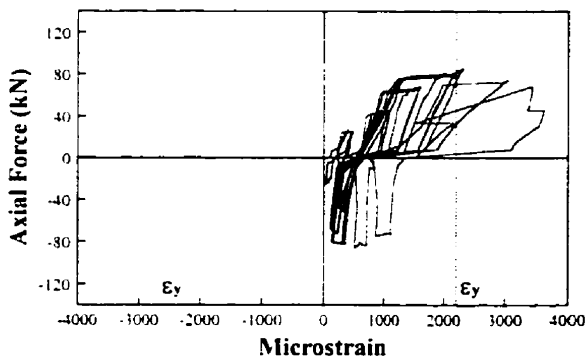


(b) Strains at Location 1, Shear response

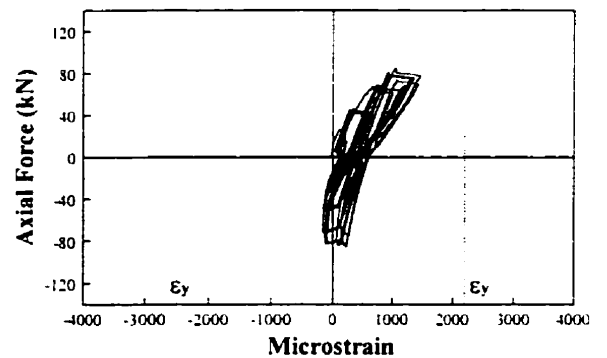


(c) Strains at Location 4, Shear response

Figure 4.46 Strains along the length of anchor for Specimen DST1 (Shear response)

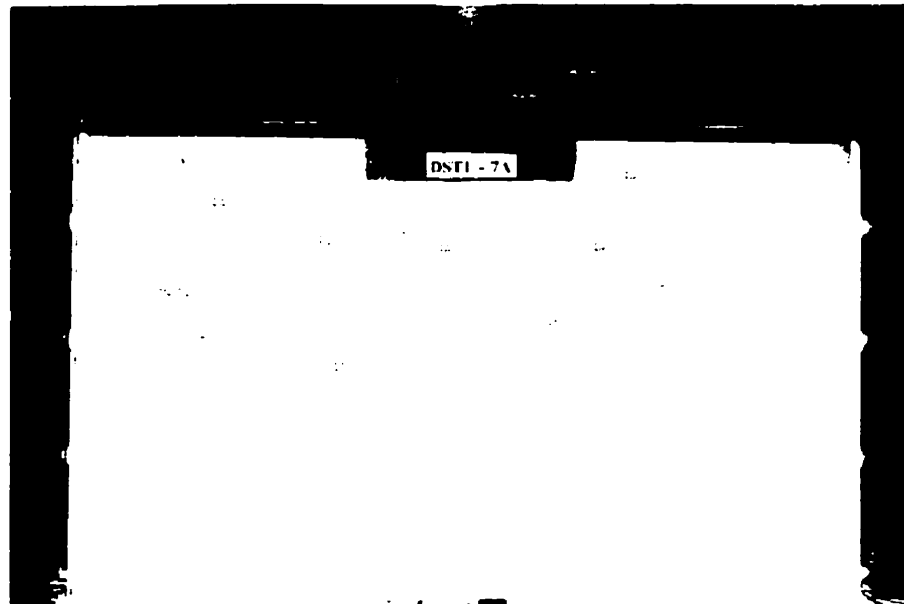


(b) Strains at Location 1, Axial response

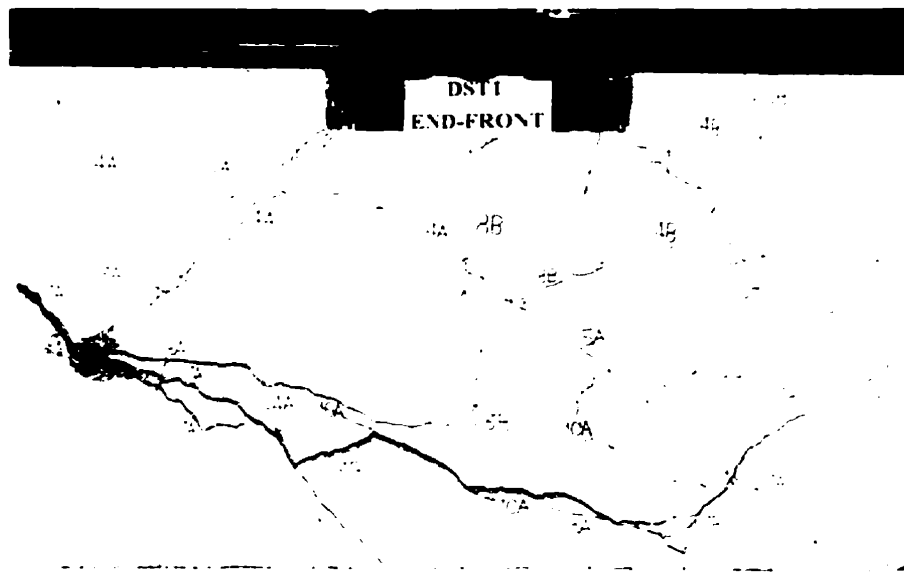


(c) Strains at Location 4, Axial response

Figure 4.47 Strains along the length of anchor for Specimen DST1 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}}=95 \text{ kN}$, $P_{\text{axial}}=68 \text{ kN}$



(b) Connection after failure, $P_{\text{ult. shear}}=120 \text{ kN}$, $P_{\text{ult. axial}}=75 \text{ kN}$

Figure 4.48 Photographs of Specimen DST1

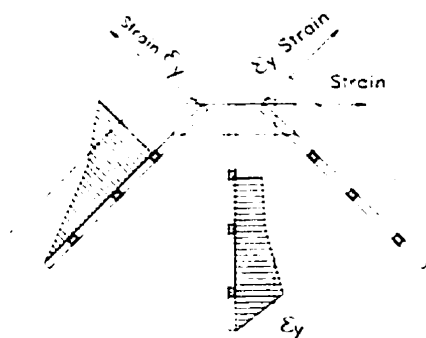


Figure 4.49 Strain distribution along the length of bar prior to failure of Specimen DST1

4.3.5 Specimen DST2

The loading history (T/V ratio = 1.0) and the associated displacements are presented in Table 4.10. The load- deflection response and the loading histories for the shear and the axial loads are shown in Figures 4.50 and 4.51, respectively. The strains along the length of the steel anchor bar for shear and axial load are shown in Figures 4.52 and 4.53, respectively. The photographs of the specimen at the yielding level and after failure are shown in Figures 4.54(a) and 4.54(b), respectively. The profile of the strain with the distance along the length of anchor bar at the ultimate load is shown in Figure 4.55. The following important behavioural aspects were noted, based on the observation during the experiment and from the analysis of the experimental results:

Cycles 1-3, $V=T=30$ kN The load-displacement curves were linear and the response was elastic with no cracks.

Cycles 4-6, $V=T=60$ kN Small splitting cracks appeared along both tension and compression legs and one diagonal crack formed in the tension zone. The slope of the load-displacement curves changed, especially for the shear response due to cracking. The area under the loops was small showing very little energy being dissipated. The maximum crack width was 0.08 mm.

Cycles 7-9, $V=T=85$ kN The reinforcing anchor bar yielded at Location 1 (Figures 4.52 and 4.53) and the horizontal and axial displacements (Δ_y) were 1.2 mm and 1.0 mm, respectively. More cracks appeared in the tension zone [see Figure 4.54(a)]. The maximum splitting and diagonal crack widths were 0.2 mm and 0.6 mm, respectively. During the positive loading cycle (shear and tensile axial loading), the hysteresis loops were large while for the negative load cycle (shear and compression loadings), the loops were very narrow (see Figures 4.50 and 4.51). The energy dissipated by the shear forces was greater than the energy dissipated by the axial response, which was very small.

Cycles 10-12, $V=T=105$ kN This stage was implemented using displacement control with the maximum displacement equal to $2\Delta_y$. The horizontal and axial displacements were 2.4 mm and 2.1 mm, respectively. The steel bar yielded at Location 2 (Figures 4.52 and 4.53). During the following cycles of this load stage, the slope of the load-deflection responses kept on decreasing and the displacements increased significantly [see Figure

4.51(b)]. This indicates that the stiffness and strength of the connection decreased significantly due to the cyclic loading. The area under the hysteresis loops increased, especially in the positive direction, showing the capability of the connection to dissipate energy through development of yielding along the reinforcing bar, propagation of cracks and crack widening over the specimen. The width of diagonal crack at the end of tension leg increased significantly to 4.0 mm.

Cycles 13-14 $V=100\text{ kN}$ $T=90\text{ kN}$ This stage involved using displacement control with the maximum displacement equal to $5\Delta_y$. This was chosen because the specimen had undergone large displacements in the previous cycles. The horizontal and axial displacements were 5.2 mm and 5.5 mm, respectively. The area under the loops for both the shear and the axial responses increased significantly, resulting in large energy dissipation throughout the cyclic loading (see Figures 4.52 and 4.53). During the last cycle in order to keep the displacement equal to $5\Delta_y$, the shear and axial loads were reduced to 85 kN and 75 kN, respectively. This was associated with a significant reduction in the stiffness of the connection.

Cycle 15 $V=85\text{ kN}$ $T=75\text{ kN}$ The connection underwent large displacements and it finally failed due to the concrete deterioration [cracking in the tension zone and crushing around the steel angle, Figure 4.54(b)]. The ultimate horizontal and axial displacements were 8.9 mm for both of them, giving a displacement ductility ratio of 8.4.

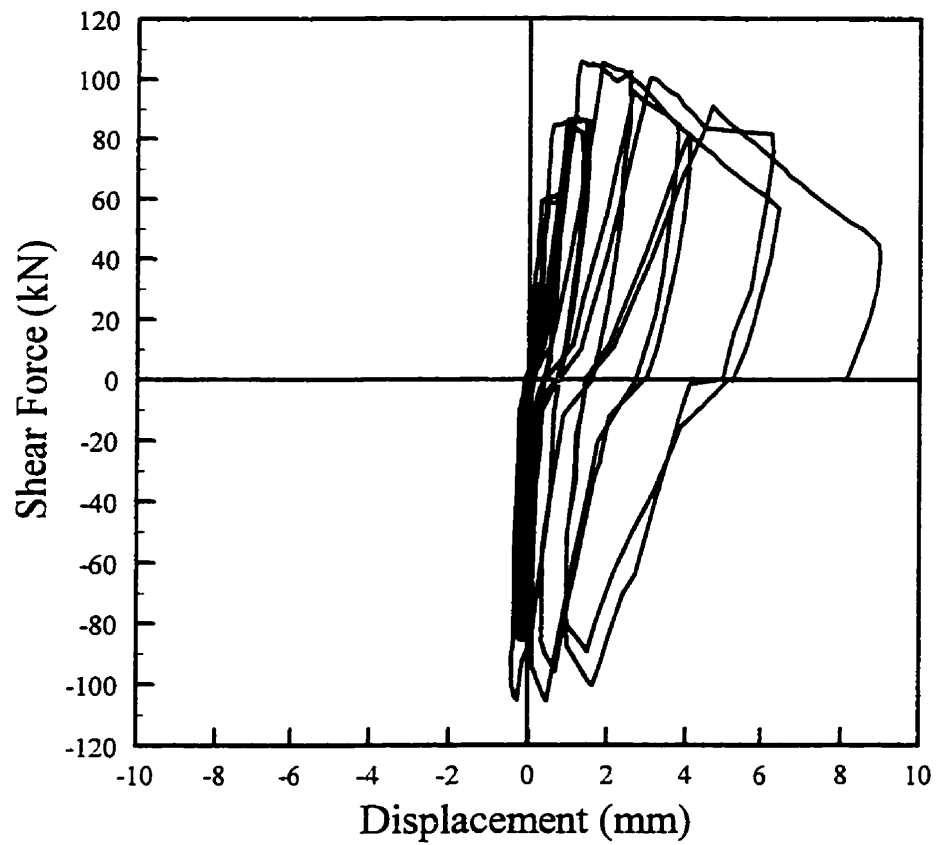
The distribution of strains along the anchor leg at the ultimate load shows that more than half of the reinforcing bar length in the tension had yielded (Figure 4.55). There was no data recorded for the compression leg because the strain gauges were lost during the casting of the specimen. No yielding was noted in the middle leg. The maximum strength of 105 kN was achieved for both shear and axial loading. The final mode of failure was due to complete opening of the diagonal crack at the end of the tensile leg and crushing of the concrete around the steel angle [see the photograph in Figure 4.54(b)]. The failure mode was close to that for the Specimen DST1. Based on the experimental results, under combined cyclic axial and shear ($T/V=1.0$), the strength of the connection reduced to about 55% and 58% of the values for pure cyclic shear and

pure cyclic axial strengths, respectively. The hysteresis response showed large loops, especially beyond the general yielding and appearance of diagonal cracks.

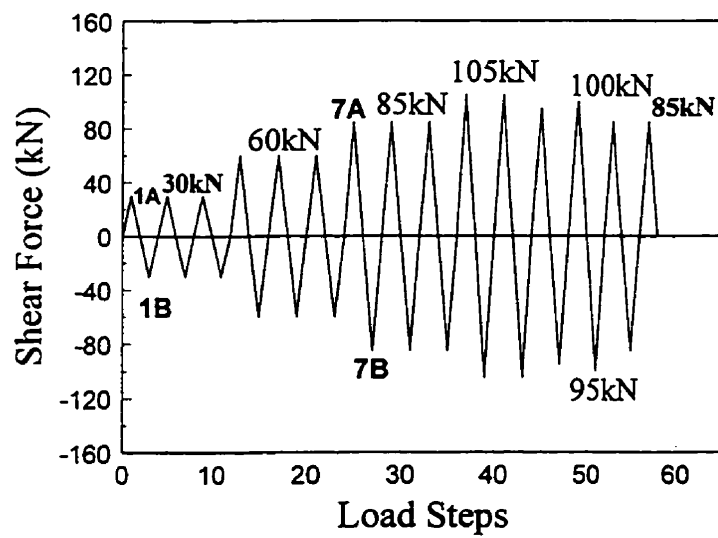
Table 4.10 Summary of loading history for Specimen DST2

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.3 , -0.03	±30	+0.1 , -0.01	±30
Cracked	4A - 6B	+0.7 , -0.08	±60	+0.5 , -0.01	±60
±1 Δ y	7A - 9B	+1.5 , -0.3	±85	+1.2 , +0.07	±85
±2 Δ y	10A - 12B	+4.1 , +0.6	±105	+4.1 , +1.0	±105
±5 Δ y	13A -14B	+6.2 , +1.2	±100	+6.9,+2.3	±90
8.4 Δ y , Failure	15A	+8.9	+85	+8.9	+75

* Maximum displacement at the peak load of the last cycle

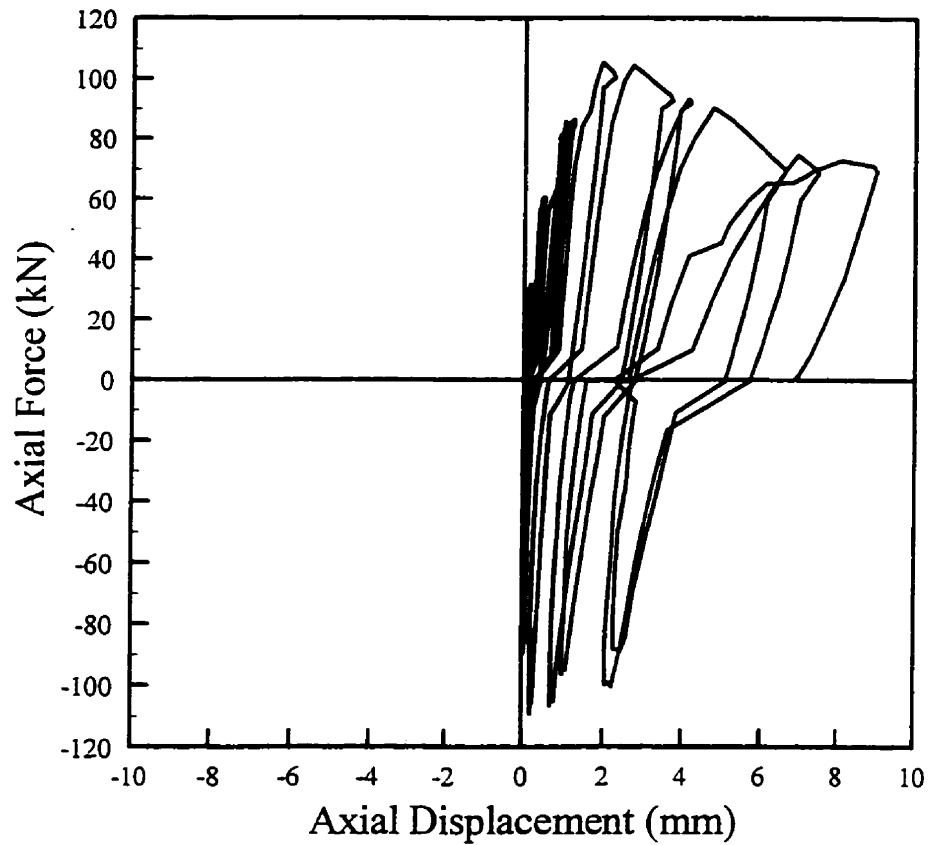


(a) Hysteresis response

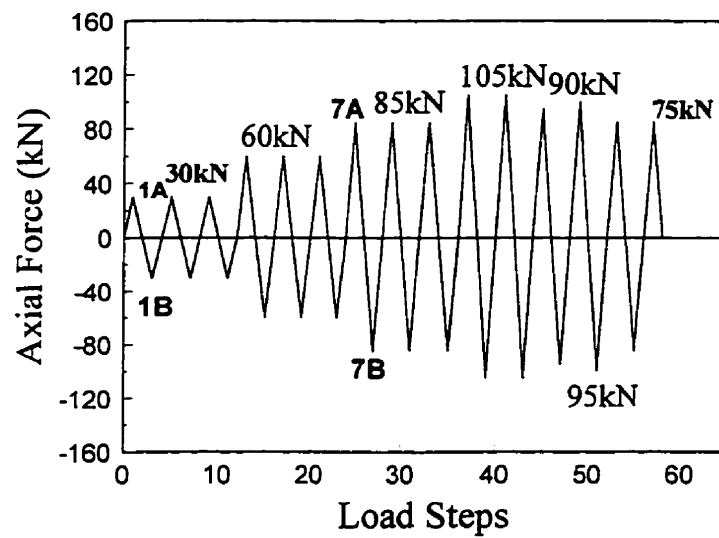


(b) Loading history

Figure 4.50 Hysteresis shear response and loading history for Specimen DST2

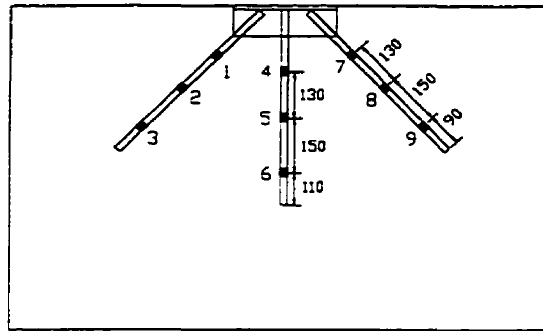


(a) Hysteresis response

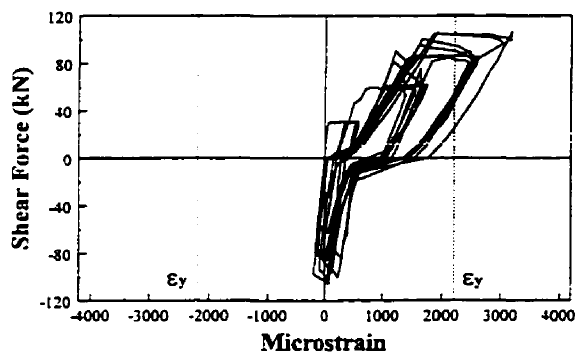


(b) Loading history

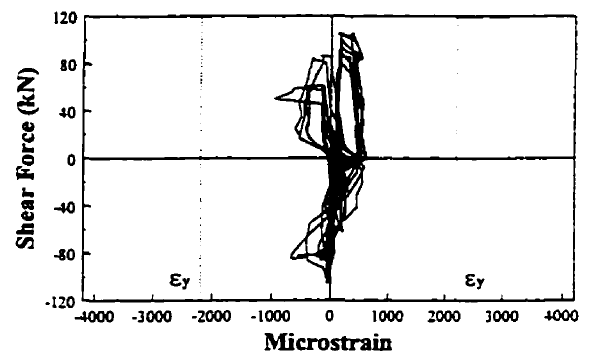
Figure 4.51 Hysteresis axial response and loading history for Specimen DST2



(a) Location of strain gauges

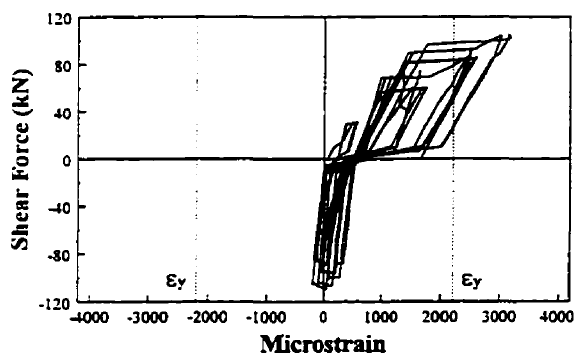


(b) Strains at Location 1, Shear response

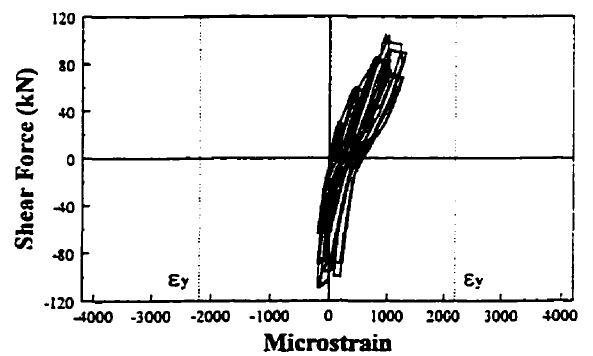


(c) Strains at Location 7, Shear response

Figure 4.52 Strains along the length of anchor for Specimen DST2 (Shear response)

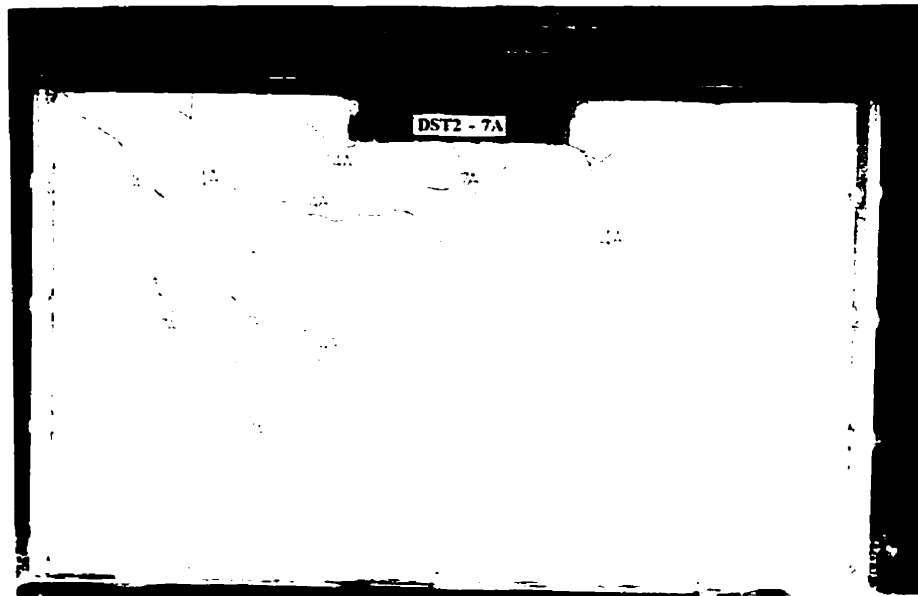


(b) Strains at Location 1, Axial response

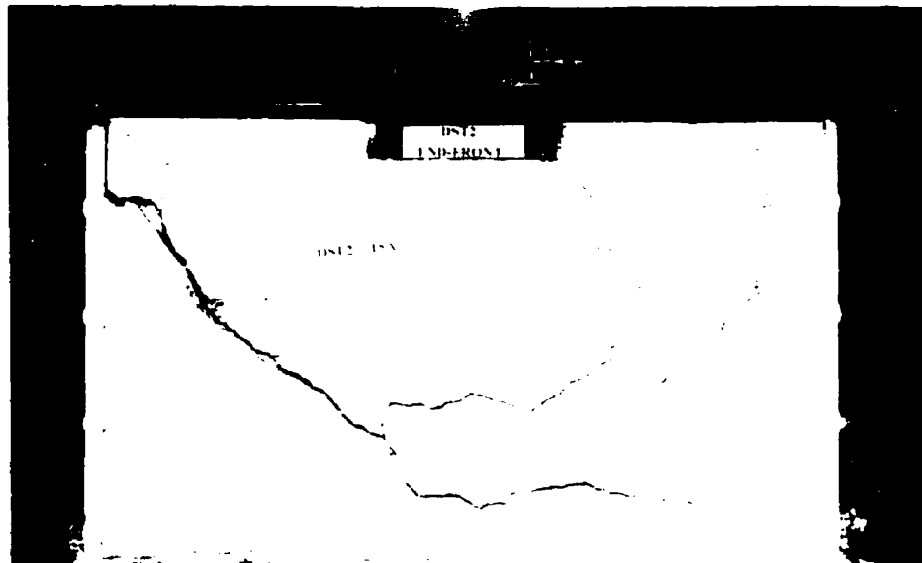


(c) Strains at Location 7, Axial response

Figure 4.53 Strains along the length of anchor for Specimen DST2 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}} = P_{\text{axial}} = 85 \text{ kN}$



(b) Connection after failure, $P_{\text{ult. shear}} = 85 \text{ kN}$ $P_{\text{ult. axial}} = 85 \text{ kN}$

Figure 4.54 Photographs of Specimen DST2

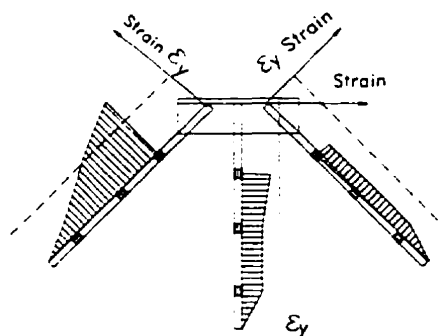


Figure 4.55 Strain distribution along the length of bar prior to failure of Specimen DST2

4.3.6 Specimen DST3

Table 4.11 summarizes the loading history (T/V ratio = 0.3) and the associated displacements. Figures 4.56 and 4.57 show the hysteresis response and loading histories for the shear and the axial loading. The measured strain in the steel anchor bar for shear and axial forces are presented in Figures 4.58 and 4.59, respectively. Figure 4.61 shows the distribution of strains along the length of steel bar at the ultimate load. Figures 4.60(a) and 4.60(b) show photographs of the test specimen at the yielding stage and after failure, respectively. The following behavioural aspects were noted during testing and analysis of the experimental results:

Cycles 1-3 $V=45$ kN $T=15$ kN The load-displacement curves were linear and there was no cracking. The behaviour was basically elastic.

Cycles 4-6 $V=90$ kN $T=27$ kN Small splitting crack appeared along the anchor bar legs and a small diagonal crack in the tension zone mostly due to the shear force. The slopes of the load-deflection curves changed and this change was more pronounced for the shear response (Figures 4.56 and 4.57). The area under the hysteresis loops was very small indicating very little energy dissipation. The displacements were also very small.

Cycles 7-9 $V=145$ kN $T=45$ kN The first yielding of the steel bar occurred at Location 1 along the tension leg (Figures 4.58 and 4.59). The horizontal and the axial displacements were 1.2 mm and 1.1 mm, respectively. Due to yielding of the steel bar, several additional diagonal cracks formed in the tension zone and one splitting crack appeared along the middle-leg [see Figure 4.60(a)]. The anchor bar yielded at Location 4 during the negative loading cycle. The maximum splitting and diagonal crack widths were 0.33 mm and 1.9 mm, respectively. The area under the axial hysteresis loops were very small, however, it was noticeable for the shear response.

Cycles 10-12 $V=165$ kN $T=50$ kN This stage of loading was implemented using displacement control with the maximum displacement equal to $2\Delta_y$. The horizontal and the axial displacements were 3.5 and 2.8 mm, respectively. During the last cycle of this stage, the displacements increased significantly despite a decrease in the shear force to 155 kN. The area under the loops increased significantly due to the extensive opening of the diagonal cracks and the development of yielding along the bar (general yielding),

which indicate energy dissipation capability of the connection. At the end of this load stage, the maximum crack widths were 0.6 mm and 0.5 mm for the splitting and the diagonal cracks, respectively.

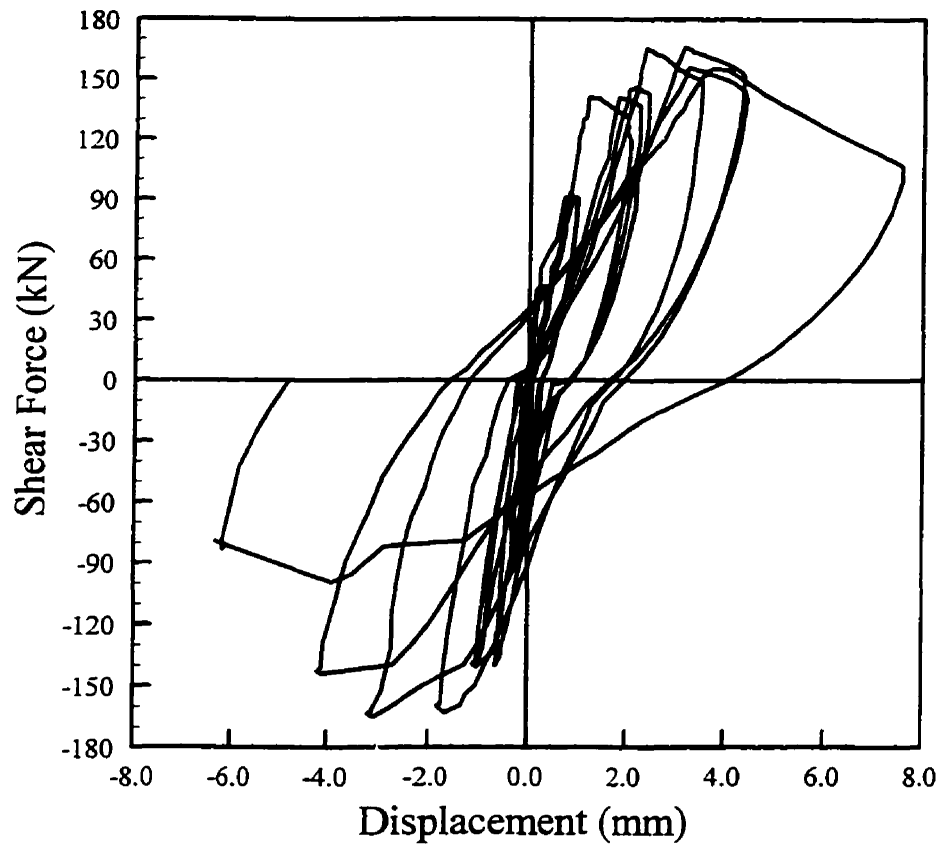
Cycle 13 $V=155$ kN $T=25$ kN The ultimate strength of connection was achieved at this stage. As the shear and the axial forces of 155 kN and 25 kN, respectively, were applied, most of the diagonal cracks opened and the specimen underwent large displacements (Figures 4.56 and 4.57). The connection could not resist any more load, resulting in collapse of the connection [Figure 4.60(b)]. During the last cycle, the area under the hysteresis loops increased very rapidly, see Figures 4.56 and 4.57, indicating a significant in energy dissipation. The ultimate horizontal and axial displacements were 7.6 mm and 5.6 mm, respectively, giving a displacement ductility ratio of 4.

The profile of strains along the anchor leg at the ultimate load shows that more than one third of the tension leg had yielded (Figure 4.61). The mode of failure was mostly due to opening of the diagonal crack at the end of the reinforcing legs [see Figure 4.60(b)]. The failure mode was similar to that for Specimen DS. The hysteresis response shows large and round loops after general yielding of the reinforcing anchor and the notable opening of the diagonal cracks, which indicates the connection energy dissipation capability. The experimental results show that when 30% of the shear force was applied simultaneously in the axial direction in a cyclic manner, the strength of connection reduced to about 87% of that for pure cyclic shear.

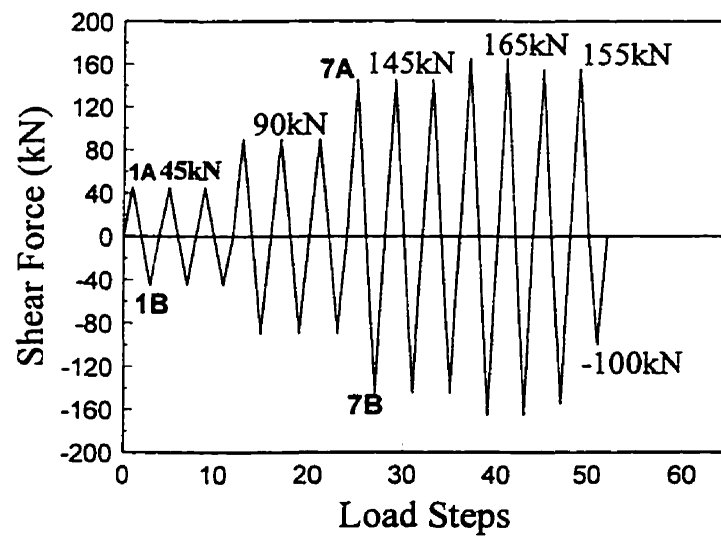
Table 4.11 Summary of loading history for Specimen DST3

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.3 , -0.02	±45	+0.1,+0.01	±15
Cracked	4A - 6B	+1.0 , -0.31	±90	+0.4 , +0.01	±27
±1 Δ_y	7A - 9B	+2.4 , -1.1	±145	+1.7 , +0.2	±45
±2 Δ_y	10A - 12B	+4.3 , -4.1	±165	+5.3 , +3.9	±50
±4 Δ_y ,Failure	13A	+7.6	+155	+5.6	+25

* Maximum displacement at the peak load of the last cycle

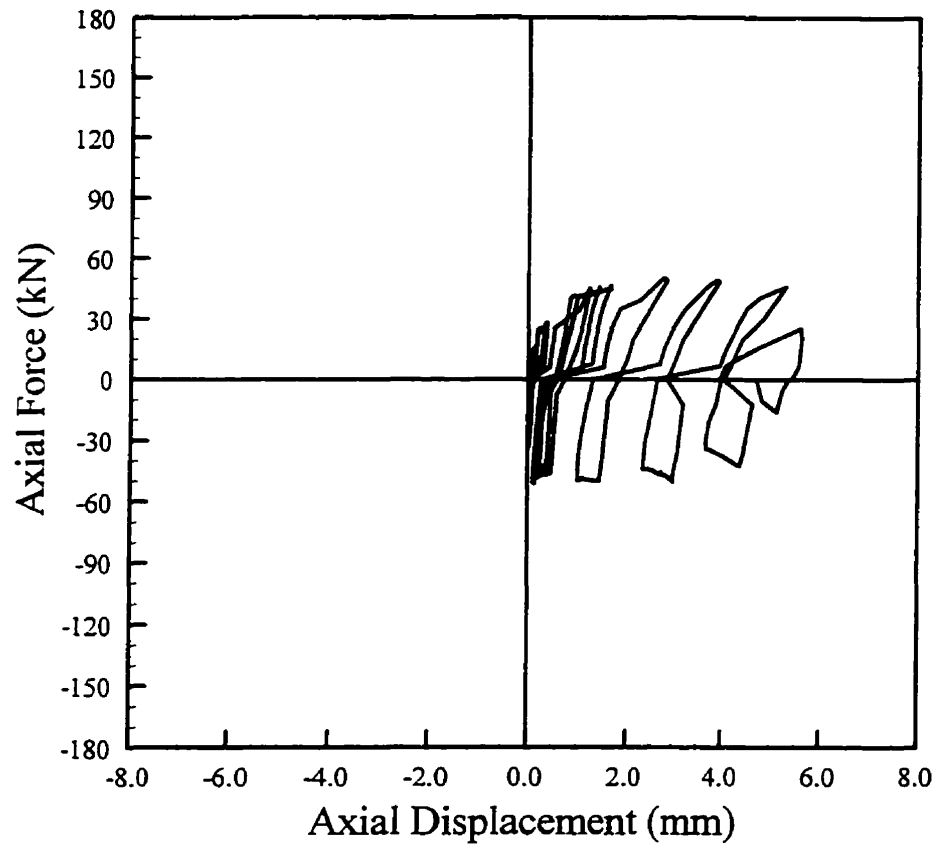


(a) Hysteresis response

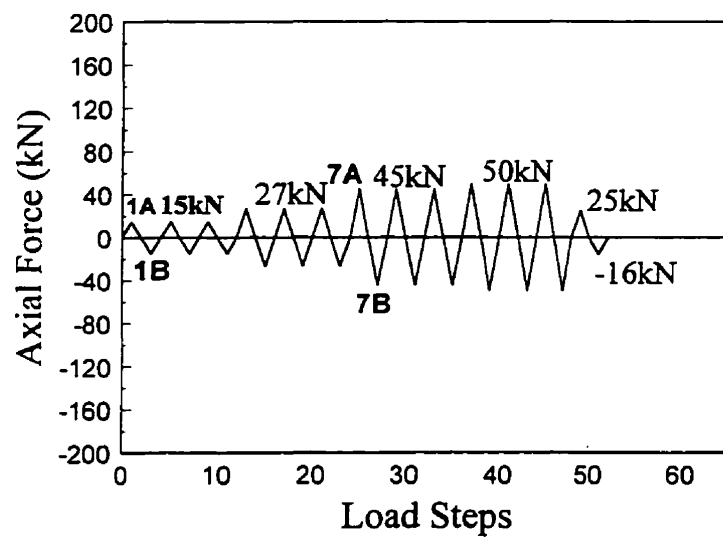


(b) Loading history

Figure 4.56 Hysteresis shear response and loading history for Specimen DST3

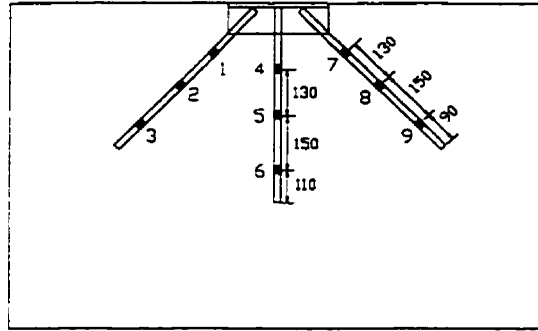


(a) Hysteresis response

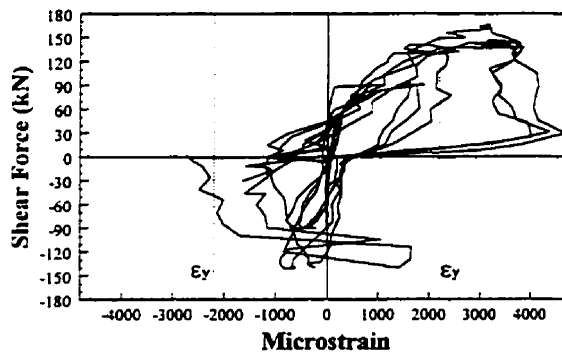


(b) Loading history

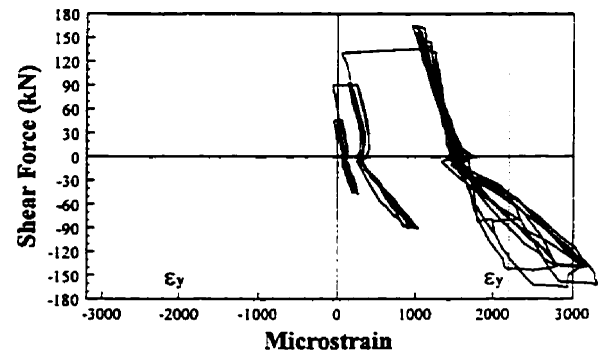
Figure 4.57 Hysteresis axial response and loading history for Specimen DST3



(a) Location of strain gauges

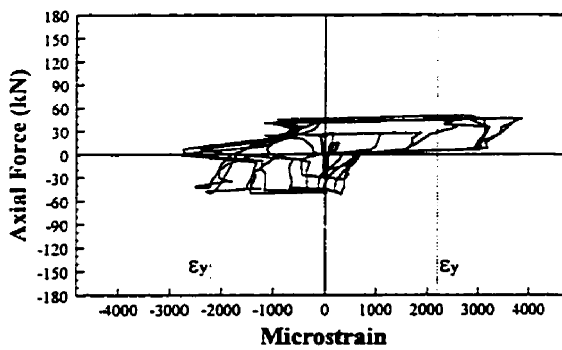


(b) Strains at Location 1, Shear response

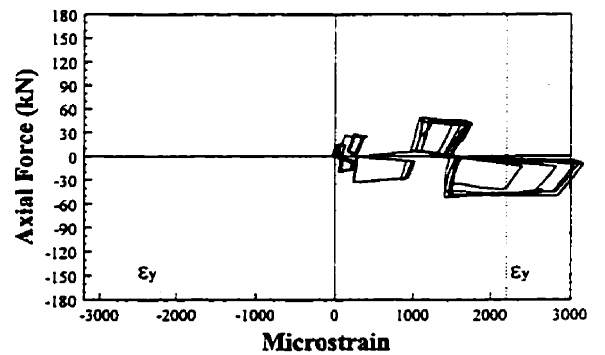


(c) Strains at Location 7, Shear response

Figure 4.58 Strains along the length of anchor in Specimen DST3 (Shear response)

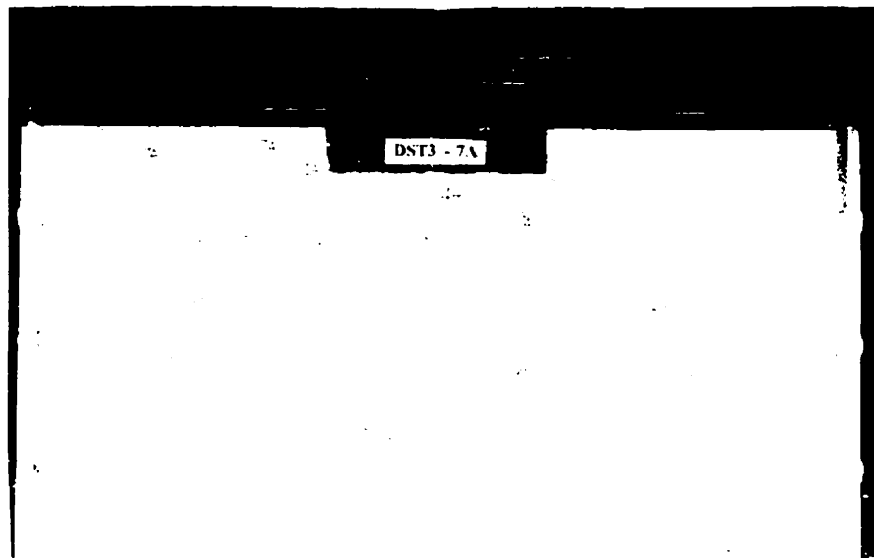


(b) Strains at Location 1, Axial response

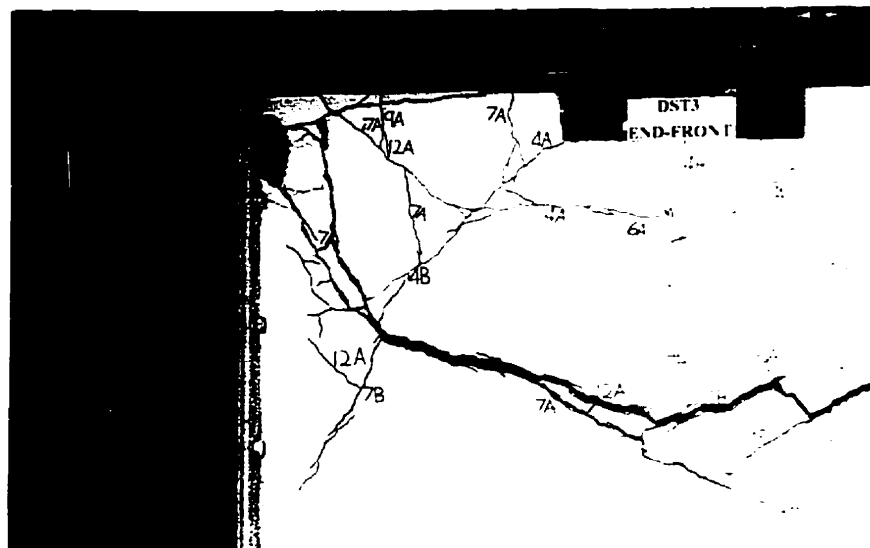


(c) Strains at Location 7, Axial response

Figure 4.59 Strains along the length of anchor in Specimen DST3 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}} = 145 \text{ kN}$, $P_{\text{axial}} = 45 \text{ kN}$



(b) Connection after failure, $P_{\text{ult, shear}} = 155 \text{ kN}$, $P_{\text{ult, axial}} = 25 \text{ kN}$

Figure 4.60 Photographs of Specimen DST3

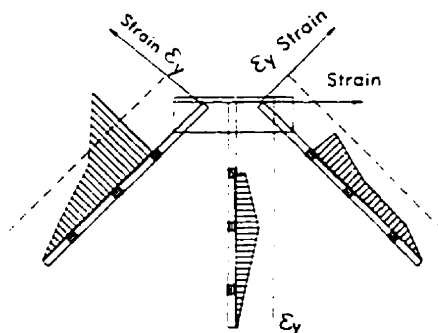


Figure 4.61 Strain distribution along the length of bar prior to failure of Specimen DST3

4.3.7 Interaction Diagram for Series D

For design of precast floor and roof systems , it is necessary to know the strength of the connections under various combinations of shear and axial forces induced during an earthquake.

The general behaviour and the ultimate capacities of the specimens tested in the Series D under the different combination of cyclic shear and axial loadings were reported in the previous sections (Sections 4.3.2 to 4.3.6). From the results obtained, and as summarized in Table 4.12, the normalized maximum cyclic shear strengths, expressed as V/V_0 are plotted versus the normalized maximum cyclic axial strength ratios, T/T_0 , in Figure 4.62. An interaction equation has been fitted to the experimental results using the best fit program, and the following interaction equation was derived:

$$\left(\frac{V}{V_0}\right) + \left(\frac{T}{T_0}\right)^{1.5} \leq 1.0 \quad (2)$$

where V is the applied shear force, V_0 is the maximum force that the connection can resist in pure cyclic shear, T is the applied axial force, and T_0 is the maximum axial force that can be resisted by the connection. The material resistance factors must be included to adopt these equation for design purposes (see Chapter 7).

It is noted from Figure 4.62 that the strength of the connection under the cyclic shear force decreases when the cyclic axial force is applied simultaneously. This is because the axial forces in addition to the shear forces cause supplementary tensile stresses at the interface of the concrete along the reinforcing bar anchors (bond stress), and, the axial loads also induced extra stresses in the concrete surrounding the steel angle. Consequently, the bond strength reduces when the shear forces are applied. Furthermore, the cracking load and the slope of the shear force-displacement curve (shear stiffness) decrease due to the application of the axial forces. Similarly, cyclic axial strength decreases when an cyclic shear force is applied simultaneously.

Table 4.12 Summary of interaction test results for Series D

Test type	Ratio of axial to shear (T/V)	Ultimate strength (kN)		Ratios	
		Shear (kN)	Axial (kN)	V/V _o	T/T _o
DS	-	190	-	1.00	0.0
DST1	0.7	130	85	0.68	0.47
DST2	1.0	105	105	0.55	0.58
DST3	0.3	165	50	0.87	0.28
DT	-	-	180	0.0	1.0

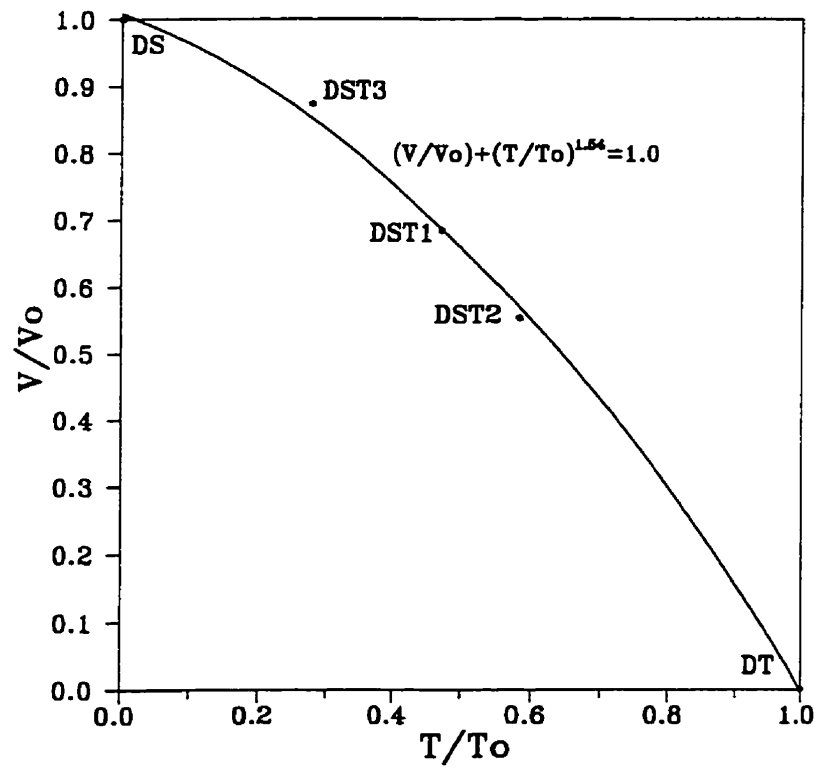


Figure 4.62 Normalized interaction curve for Series D

Chapter 5

Experimental Results - Series E

5.1 Introduction

The complete responses of six identical specimens in Series E (Specimens EMS, ES, ET, EST1, EST2 and EST3), tested to destruction under reversed cyclic loads are described in this chapter. The specimen hysteresis response under monotonically increasing shear loads and reversed cyclic shear and axial (tensile and compressive) loading, shear force-axial force interaction diagrams, total specimen displacements and the strains at two locations on the headed studs were measured.

In order to study the cyclic response of the connections to combined axial and shear forces, one specimen was tested under monotonically increasing shear load and the five remaining specimens were subjected to reversed cyclic combined shear and axial forces with different values of the cyclic axial to cyclic shear force ratios. The axial forces were applied with a delay of quarter cycle after application of the shear forces.

The following figures are presented for each of the six connection specimens:

1. Monotonically applied shear load vs. horizontal displacement (response to monotonically increasing shear load).
2. Cyclic applied load vs. horizontal displacement response (cyclic shear response).
3. Cyclic applied load vs. axial displacement response (cyclic axial response).
4. Applied load vs. measured strain in the headed stud anchors.
5. Photographs of the specimen at different load stages.
7. Shear force -axial force interaction diagrams for the connection.

5.2 Test Series E: Connection with headed studs welded to a steel plate

This connection was designed with two 1/2 in. (13 mm) diameter Nelson studs 4 in. (100 mm) long, welded to a 10 mm thick steel plate (see Figure 2.7). A comprehensive description of connection can be found in Section 2.5. The design of the connection and the determination of its ultimate shear capacity were performed based on the current design method suggested in the CPCI Metric Design Manual⁽¹⁶⁾. There were no design consideration for cyclic load.

5.2.1 Specimen EMS

Figure 5.1 shows the shear force-horizontal displacement curve for the specimen. The load versus the measured strains response in the headed studs is presented in Figure 5.2. Figure 5.3 shows photograph of the connection after failure. The following important behavioural aspects were noted during the testing and analysis of the experimental results:

Stage 1 29 kN The first cracks appeared at the end of the plate due to the crushing of the concrete in front of the plate. The load-deflection remained almost linear up to appearance of the cracks (Figure 5.1). The horizontal displacement and crack width were 0.15 mm and 0.08 mm, respectively.

Stage 2 45 kN The headed stud yielded at Location 1 (Figure 5.2), while the concrete in front of the steel plate spalled off and cracks appeared and extended along the studs. This caused a change in slope of the load-deflection curve (Figure 5.1). The measured displacement at yield and the maximum crack width along the studs were $\Delta_y = 0.42$ mm and 0.1 mm, respectively.

Stage 3 53 kN When the load was increased to the ultimate load value of 53 kN, the concrete in front of the steel plate crushed completely. This caused that entire shear force to be resisted by the studs resulting in a large increase in the strains in the studs (Figure 5.2); this was accompanied by a drop in the load to 40 kN. During this load drop, the stud at Location 2 yielded (Figure 5.2(b)) and it failed in a very short time by the shearing off of the stud. After failure of stud at Location 2, the strain at Location 1 increased significantly, as the load was carried entirely by this stud (Figure 5.2(c)). As

the load was increased further, the connection could not carry any more load and the horizontal displacement increased significantly. The first stud failed at the load of 40 kN and the connection failed due to the failure of both studs (Figure 5.3). The ultimate displacement prior to failure was 3.3 mm given a displacement ductility of 7.8.

The ultimate load achieved was 53 kN. The load-displacement response shows a large plateau after failure of the stud at Location 2. It was noted that the cracking of concrete around the studs at yield level was not significant. The mode of the failure was due to complete crushing of the concrete in front of the steel plate and shearing off of both headed studs as shown in Figure 5.3.

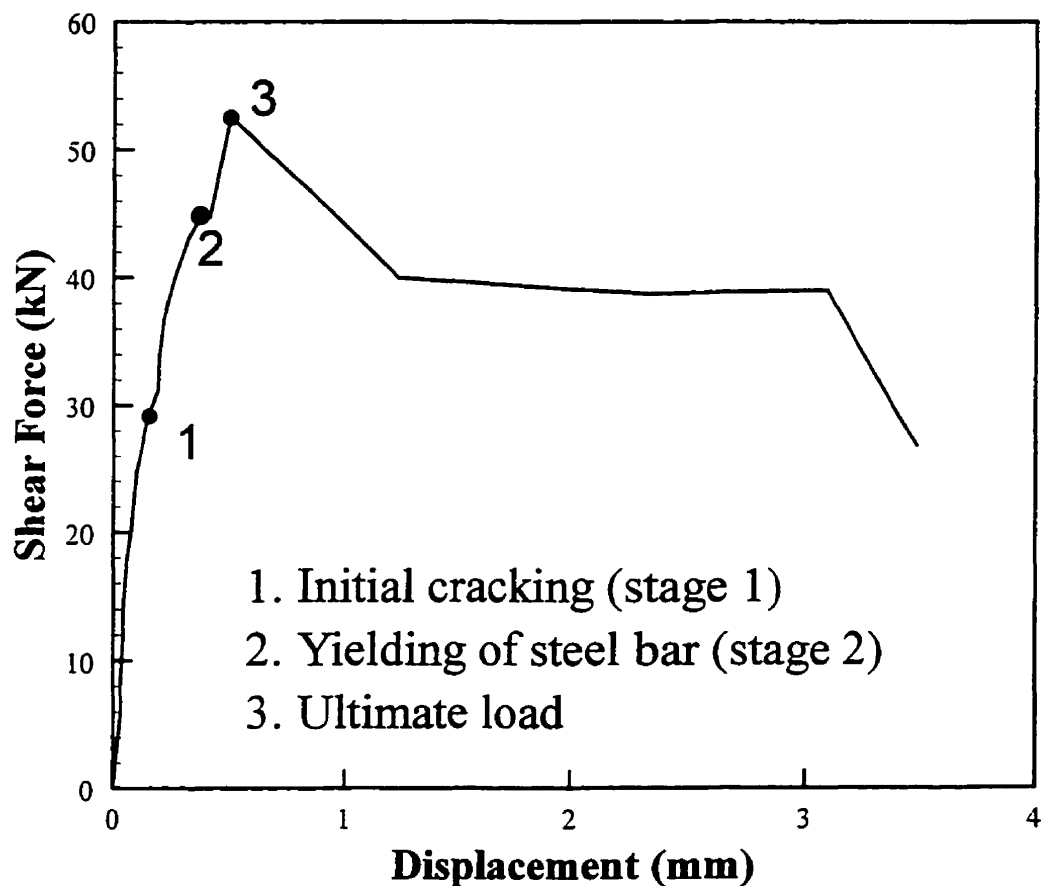
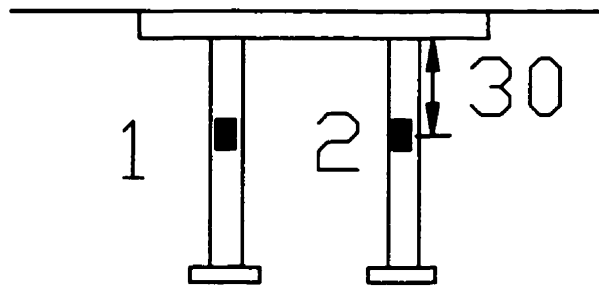
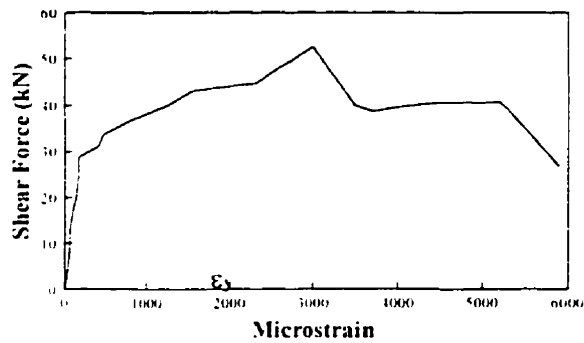


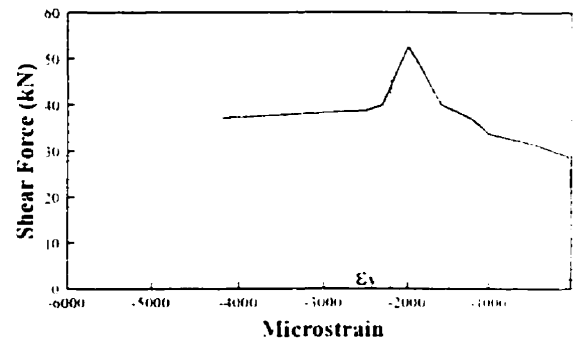
Figure 5.1 Shear force vs. horizontal displacement curve for Specimen EMS



(a) Location of strain gauges



(b) Strain at Location 1



(c) Strain at Location 2

Figure 5.2 Measured strains along the stud anchors for Specimen EMS



Figure 5.3 Photograph of Specimen EMS after failure

5.2.2 Specimen ES

The Specimen ES from Series E was subjected to reversed cyclic shear only, as summarized in Table 5.1. The loading history and the hysteresis response of the specimen are presented in Figure 5.4. The shear force versus the strains in the stud anchors response is shown in Figure 5.5. The photograph of the specimen after failure is shown in Figure 5.6. The following observations and behavioural aspects were noted during the testing and analysis of experimental results:

Cycles 1-3, 15 kN No cracks were noted during Cycles 1 and 2, however, the first crack was observed at front of steel plate during Cycle 3. The nonlinear response started due to cracking (Figure 5.4), however, the displacement was very small (Table 5.1).

Cycles 4-6, 25 kN The stud anchor at Location 1 yielded (Figure 5.5). The displacement at yield was measured to be $\Delta_y = 0.33$ mm (Figure 5.4). A diagonal crack appeared at the back of the specimen and extended toward the ends of the studs. The hysteresis loops were pinched indicating low energy dissipation. The maximum crack width and the displacement measured at the end of this load stage were 0.1 mm and 0.37 mm, respectively.

Cycles 7-9, 45 kN This stage was implemented with a displacement control of $\Delta = 2\Delta_y = 0.64$ mm. The concrete at the front of steel angle crushed and spalled off. The stud anchor at Location 2 yielded (Figure 5.5), when the shear force was applied in the negative direction (Cycle 7B). The area under the hysteresis loops did not increase significantly, showing little energy dissipation. During Cycle 8B, a crack formed along the stud at Location 2. Upon applying shear force during Cycle 9A, the stud at Location 2, subjected to compressive strain yielded. The measured crack width at the peak of last cycle was 0.1 mm.

Cycle 10, 46 kN Finally, the specimen was subjected to a load of 46 kN, when the concrete in front of the steel plate crushed completely resulting in a significant increase in the horizontal displacement. This caused failure of the stud in compression (Location 2). Due to the failure of the first stud, the load dropped to about 35 kN, while all of the shear force was carried by the second stud (Location 1), resulting in a significant increase in the stud strain (Figure 5.5). The measured displacement prior to failure of the stud was

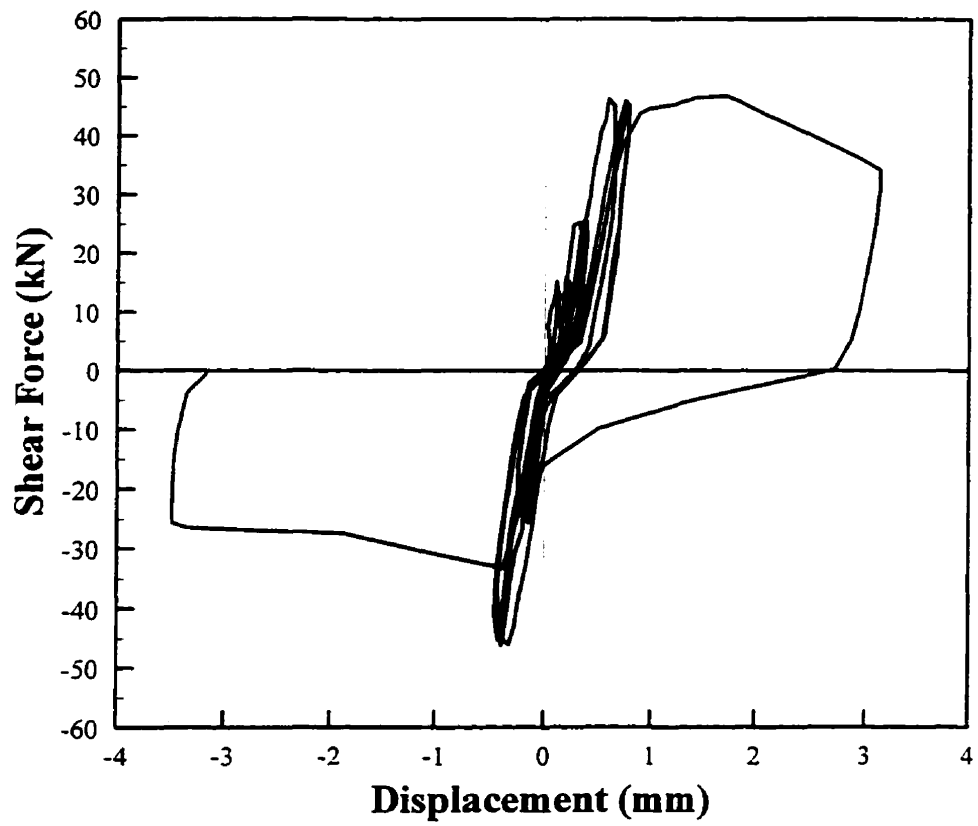
1.75 mm (Figure 5.4). When the load of -33 kN was applied in negative direction (Cycle 10B), all of the load was resisted by the remaining stud and the connection underwent a large displacement. The compressive strain at Location 1 increased significantly (Figure 5.5), and finally the connection failed because of the stud failure. The area under the last loop increased significantly as compared with the previous cycles (Figure 5.4).

The ultimate capacity of 46 kN was achieved along with the failure mode of complete deterioration of the concrete around the steel plate and failure of both headed studs (Figure 5.6). The hysteresis response consisted of large loops beyond the yielding of both studs, specially during the last cycle, showing the capability of connection to dissipate energy. The ultimate displacement prior to failure at the peak of Cycle 10A was 1.7 mm, giving a displacement ductility of 5.

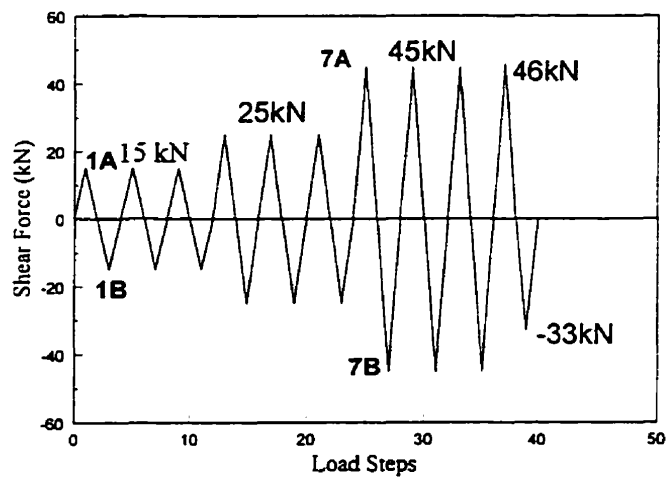
Table 5.1 Summary of loading history for Specimen ES

Cycle	Load Stages	Horiz. Displacement (mm)*	Shear Force (kN)
Elastic	1A - 2B	+0.1 , -0.1	±15
Cracking	3A - 3B	+0.2 , -0.1	±15
±1Δy	4A - 6B	+0.4 , -0.2	±25
±2Δy	7A - 9B	+0.8 , -0.4	±45
Failure,+5Δy	10A - 10B	+1.8, -1.9	+123

* Maximum displacement at the peak load of the last cycle

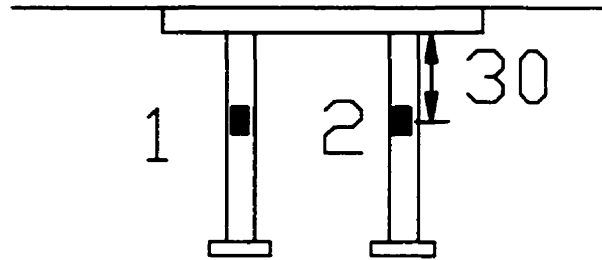


(a) Hysteresis response

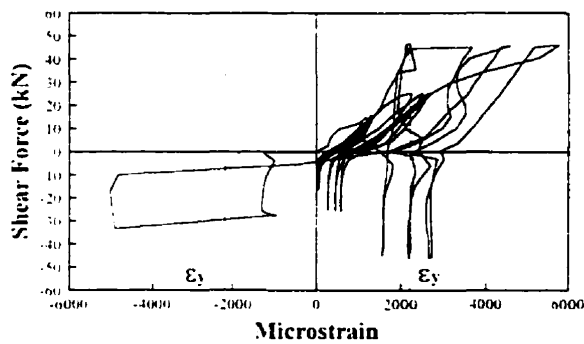


(b) Loading history

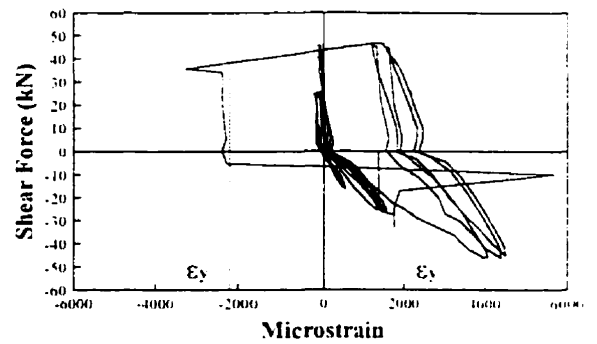
Figure 5.4 Hysteresis response and loading history of Specimen ES



(a) Location of strain gauges



(b) Strain at Location 1



(c) Strain at Location 2

Figure 5.5 Measured strains along the stud anchors for Specimen ES

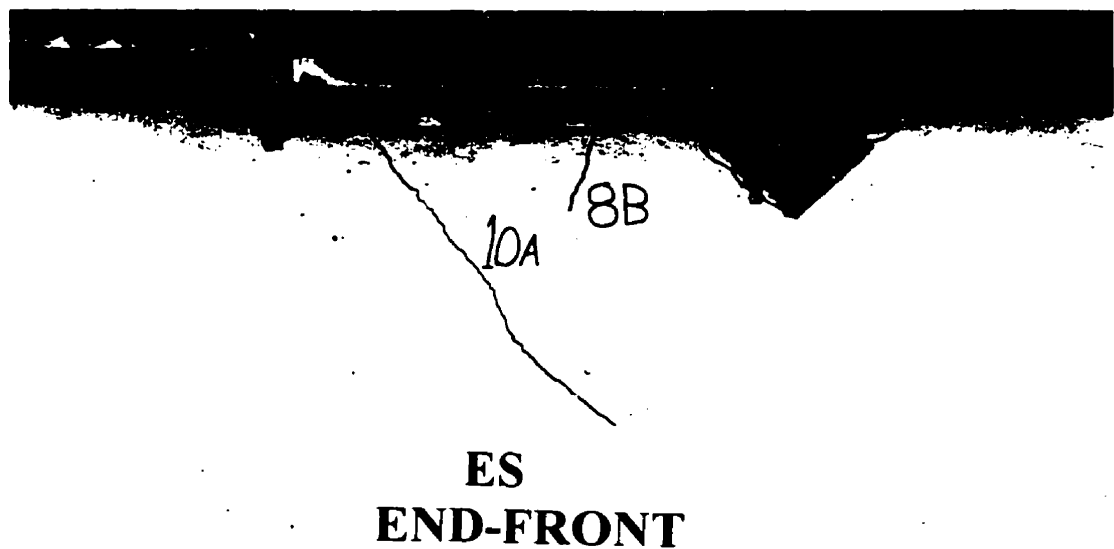


Figure 5.6 Photograph of Specimen ES after testing

5.2.3 Specimen ET

The Specimen ET was tested under reversed axial loads only. The loading history and the displacements are presented in Table 5.2. Figure 5.7 shows the hysteresis response and the loading history of the specimen. The applied axial load versus the strain response at the different locations in the stud anchors are shown in Figure 5.8. Photograph of the connection after failure is presented in Figures 5.9. The following behavioural aspects were noted based on the observations made during the testing and analysis of the experimental results:

Cycles 1-3, 10 kN The connection response during the first three cycles was basically elastic (Figure 5.7), and no cracking occurred. The displacement was small (Table 5.2).

Cycles 4-6, 23 kN First small crack was observed at the interface of steel plate and the concrete, however, the slope of the load-deflection curve did not change and the response remained linear elastic. The displacements were small (Table 5.2) and the area under the hysteresis loops was negligible (Figure 5.7).

Cycles 7-9, 28 kN The headed stud anchors yielded at both Locations 1 and 2 (Figure 5.8) which caused a change in the slope of the load-deflection curve and a decrease in the connection stiffness (Figure 5.7). The displacement at yield was $\Delta_y = 1.0$ mm. The concrete-steel plate interface crack extended towards the corner of steel plate with a measured crack width of 0.4 mm. The area under the hysteresis loops increased indicating the considerable energy dissipation through the development of yielding. The displacements during the application of the compressive load (negative direction) were not recovered completely due to partial closing of interface crack.

Cycles 10-12, 31 kN This stage was implemented using displacement control with the deflection equal to $2\Delta_y$. The measured displacement was $2\Delta_y = 2.0$ mm. No more cracks appeared while the interface crack widened to a width of about 1.9 mm. The stiffness of the connection kept on decreasing and the area under loops increased significantly showing an increase in the dissipated energy.

Cycles 13-14, 32 kN This stage of loading was also implemented using displacement control with the deflection equal $3\Delta_y = 3.0$ mm. The interface crack extended and the concrete surrounding the steel plate corners was broken away. The interface crack opened

to a width of 2.9 mm and the steel plate move axially and it got detached from concrete. The stiffness of the connection kept on decreasing while the area under the hysteresis loops increased significantly (Figure 5.7). The strains in the studs increased very significantly (Figure 5.8). During the negative loading, the compressive strain at Location 2 exceeded the stud yield strain (Figure 5.8). This was accompanied by the crushing of the concrete under the steel plate.

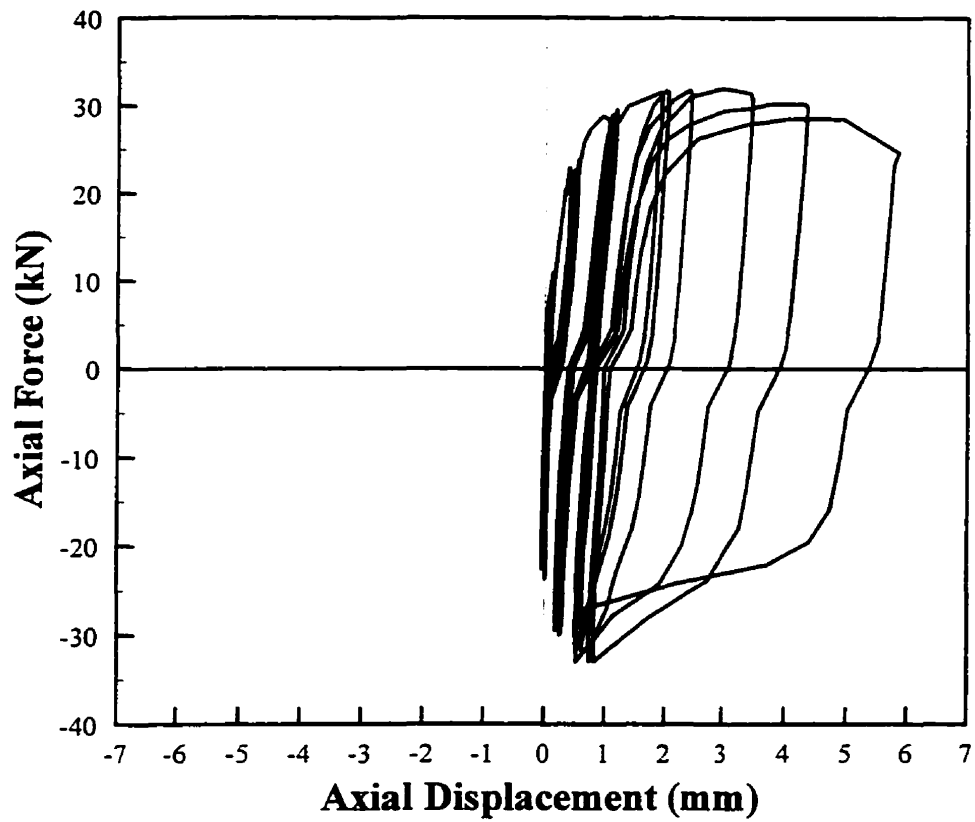
Cycle 15, 28 kN As the connection was subjected to a load value of 28 kN, the stiffness of the connection reduced very significantly, while the connection underwent a large displacement. The load dropped to about 23 kN. The crack width was 3.5 mm. It was observed that the stud anchors slipped out slightly. During the negative loading, as a load of -28 kN was applied, the slope of load-deflection curve decreased significantly. Upon application of the tensile axial force during the following cycle, the stud anchors were pulled out (Figure 5.9) and the connection could not resist any more load. The ultimate displacement during Cycle 15 was measured to be 5.8, given displacement ductility $\mu=5.8/1.0=5.8$.

The maximum strength achieved was 32 kN. The mode of failure was due to the pull out of the stud anchor and breaking away of the concrete surrounding the steel plate (Figure 5.9). The hysteresis response showed large and rounded loops which was due to developing of yielding in the studs and the slipping of the stud anchors.

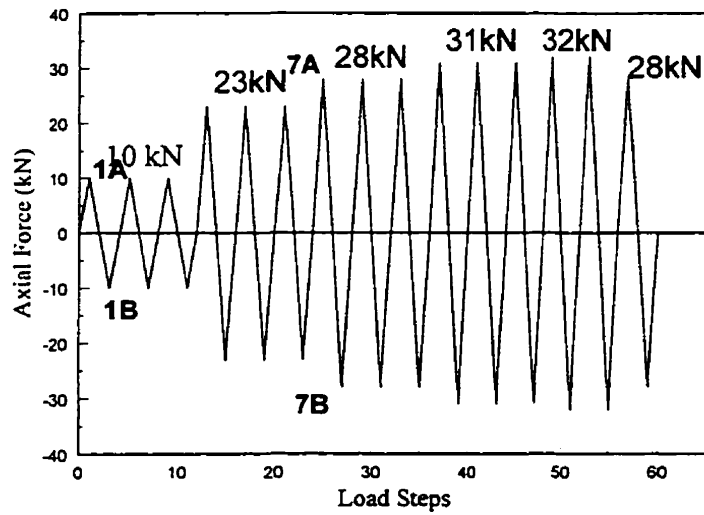
Table 5.2 Summary of Loading History for Specimen ET

Cycle	Load Stages	Axial Displacement (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.2 , -0.01	±10
Cracked	4A - 6B	+0.5 , +0.03	±23
±1Δy	7A - 9B	+1.2 , +0.3	±28
±2Δy	10A - 12B	+2.5 , +0.7	±31
±3Δy	13A - 14B	+4.4 , +0.6	±32
Failure, 5.8Δy	15A - 15B	+5.8, +0.5	±28

* Maximum displacement at the peak load of the last cycle

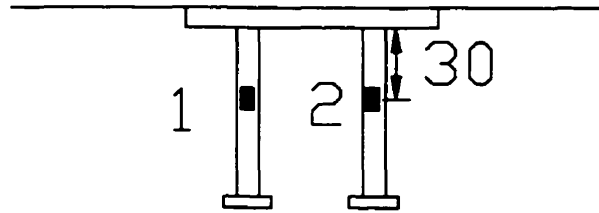


(a) Hysteresis response

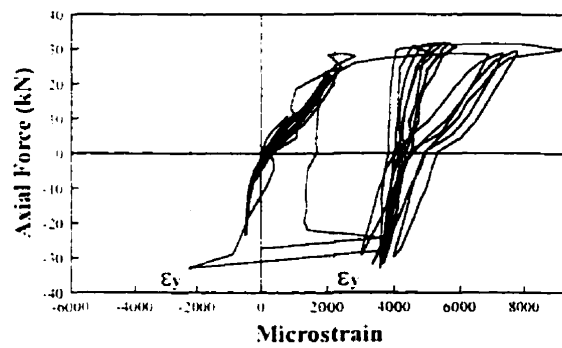


(b) Loading history

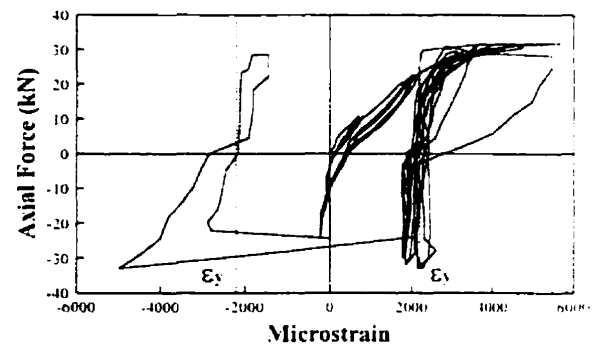
Figure 5.7 Hysteresis response and loading history for Specimen ET



(a) Location of strain gauges



(b) Strain at Location 1



(c) Strain at Location 2

Figure 5.8 Measured strains along the stud anchors for Specimen ET

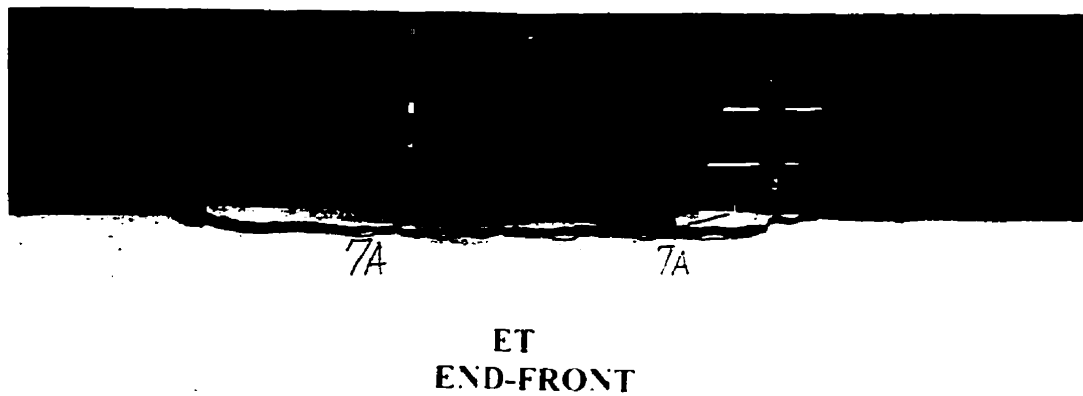


Figure 5.9 Photograph of Specimen ET at failure

5.2.4 Specimen EST1

The loading history and the accompanying result displacements for Specimen EST1 ($T/V=0.7$) are presented in Table 5.3. Figures 5.10 and 5.11 show the hysteresis curves and the loading histories for shear and axial responses, respectively. The measured strains in steel stud anchors are presented in Figures 5.12 and 5.13. The photographs of the specimen at the yield stage and after failure are shown in Figures 5.14(a) and 5.14(b), respectively. The following important notes were made based on the observations during the testing and from the analysis of the experimental results (V and T stand for shear force and axial force, respectively):

Cycles 1 - 3, $V=9\text{ kN}$ $T=7\text{ kN}$ Linear behaviour was observed for both axial and shear responses during the first steps of loading and no cracks appeared up to end of this load stage.

Cycles 4 - 6, $V=15\text{ kN}$ $T=10\text{ kN}$ Small cracks appeared at the concrete-steel plate interface, however, it did not change the slope of the load-deflection curves, and the behaviour remained elastic up to the end of this load stage (Figures 5.10 and 5.11). During the following cycles, the interface cracks extended to the corner of the steel plate. The horizontal and axial displacements were 0.13 mm and 0.11 mm, respectively (Table 5.3). The areas under the hysteresis loops were very small and the energy dissipated was negligible.

Cycle 7, $V=35\text{ kN}$ $T=22\text{ kN}$ The headed stud anchor at Location 1 yielded (Figures 5.12 and 5.13). The measured horizontal and axial displacements were 1.5 mm and 1.0 mm, respectively, giving the resultant yield displacement of $\Delta_y=1.8\text{ mm}$. Due to this yielding the interface crack widened to about 0.5 mm and the concrete at the front of steel plate spalled off (Figure 5.14(a)). The slope of load-deflection curves reduced indicating a reduction in the stiffness of the connection (Figures 5.10 and 5.11). The area under the loops increased during the positive loading (shear and axial tension), however, in the negative load direction (shear and axial compression), the loops were very narrow.

Cycle 8, $V=22\text{ kN}$ $T=9\text{ kN}$ This stage was implemented using displacement control with the deflection equal to $2\Delta_y$. The concrete cover around the steel studs started to spall off. The horizontal and axial displacements were 3.9 mm and 0.65 mm,

respectively, given the resultant displacement of $2\Delta_y=3.95$ mm. The stiffness of the connection kept on decreasing and the area under the hysteresis loops increased. The strength of the connection decreased significantly due to the reversed cyclic loading (Figures 5.10 and 5.11).

Cycle 9, $V=17$ kN $T=9$ kN

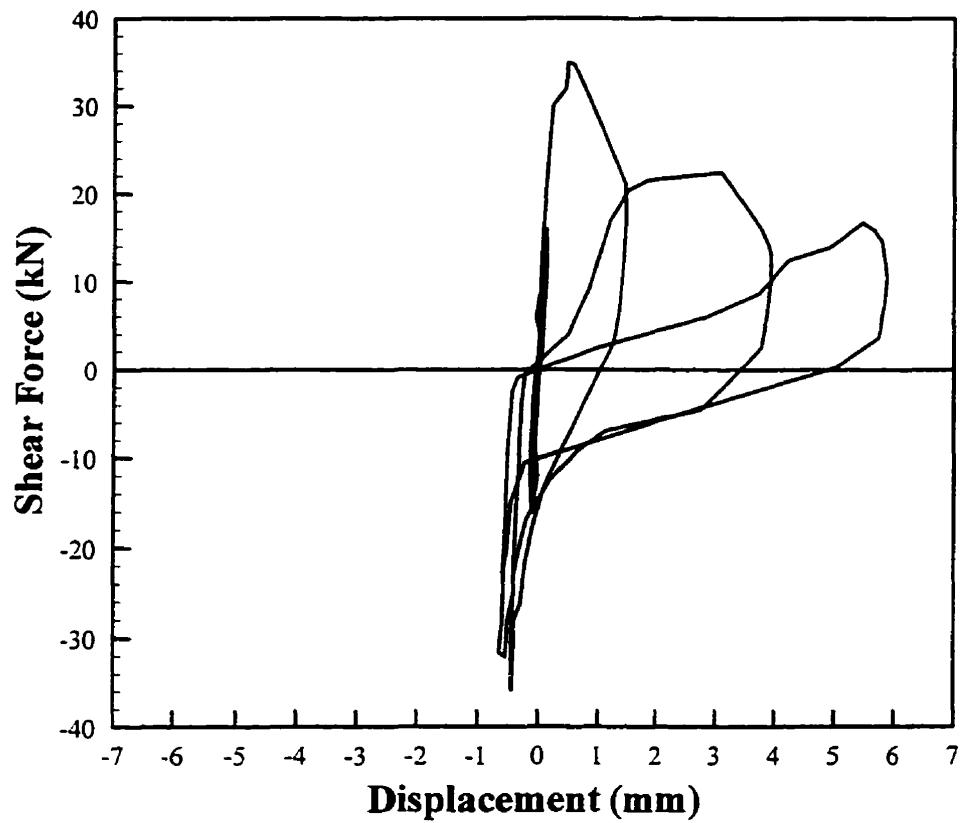
The stiffness of the connection decreased very significantly and connection underwent large displacement, specially for the shear response. the concrete cover spalled off and concrete at front plate broke away. The studs bent and the one in tension fractured due to combined shear and tensile axial stresses. The ultimate horizontal and axial displacements were 5.5 mm and 0.85 mm, respectively. The strength of the connection decreased very significantly. Upon applying the shear force in the negative direction, the other stud suddenly fractured resulting in the failure of the specimen (Figure 5.14(b)).

The maximum shear and axial strengths of 35 kN and 22 kN were achieved, given the resultant strength of 41 kN acting at an angle of 32° with respect to direction of applied shearing load. The failure mode was due to the complete deterioration of the concrete surrounding the steel plate and fracture of the stud anchors (Figure 5.14(b)). The resultant ultimate displacement was calculated to be 5.6 mm, which gives the displacement ductility ratio $\mu = 5.6/1.0 = 5.6$. The results show that under combined cyclic axial and shear loads ($T/V=0.7$), the strength of connection decreased to about 76% and 70% of those for pure cyclic shear and pure cyclic axial strengths, respectively. The hysteresis responses showed larger loops in the positive direction (shear and tensile axial load). This was mostly due to the axial load which caused large strains in the stud anchors (Figures 5.12 and 5.13).

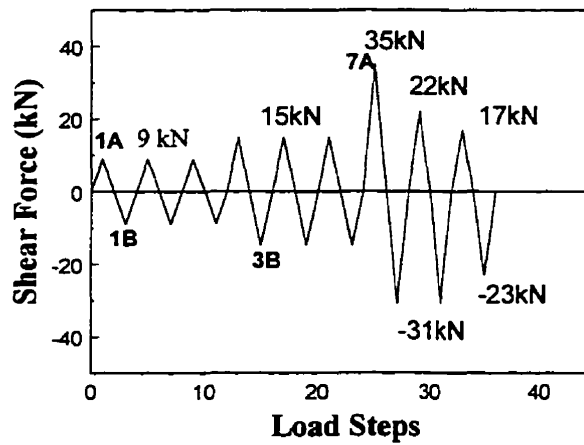
Table 5.3 Summary of loading history for Specimen EST1

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.07 , -0.07	±9	+0.03, -0.03	±7
Cracked	4A - 6B	+0.13 , -0.11	±15	+0.11 , -0.01	±10
±1Δy	7A - 7B	+1.5 , -0.44	±35	+1.0 , +0.04	±22
±2Δy	8A -8B	+3.9 , -0.63	+22 , -31	+0.65 ,+0.06	+9 , -18
±5.6Δy, Failure	9A -9B	+5.5 , -0.58	+17 , -23	+0.85	+9

* Maximum displacement at the peak load of the last cycle

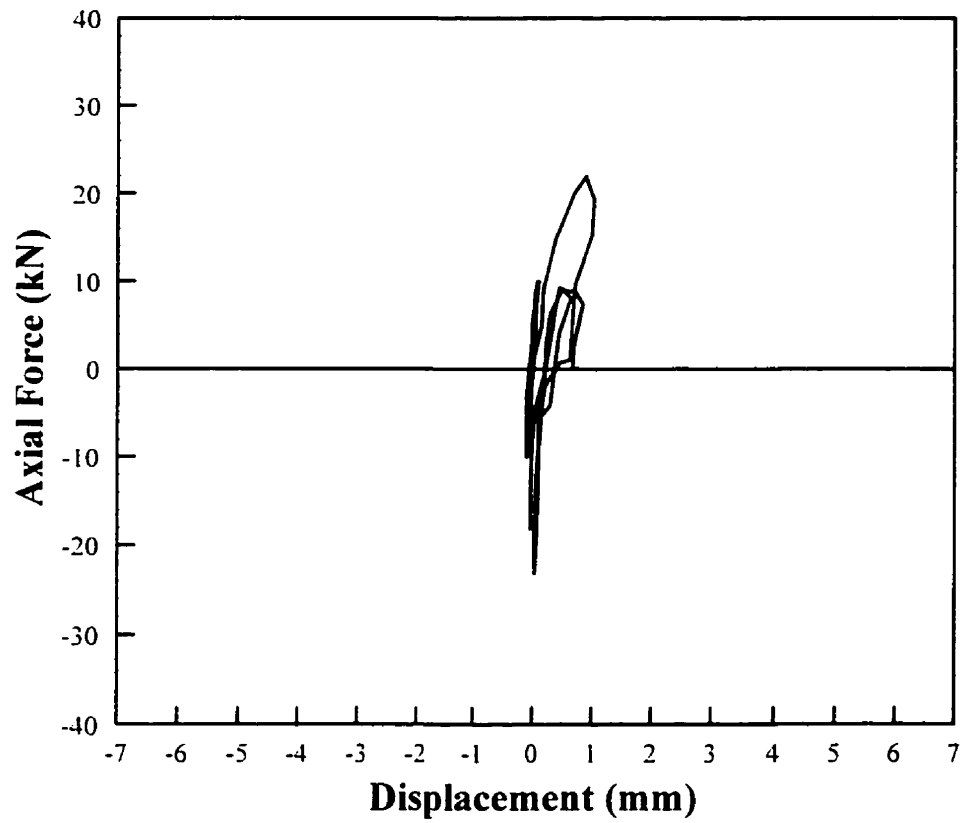


(a) Hysteresis response

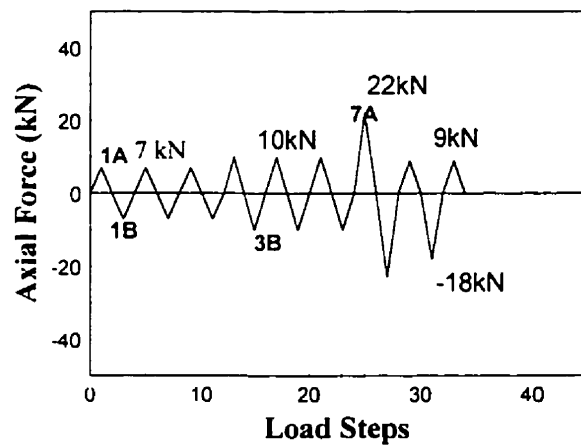


(b) Loading history

Figure 5.10 Hysteresis shear response and loading history of Specimen EST1

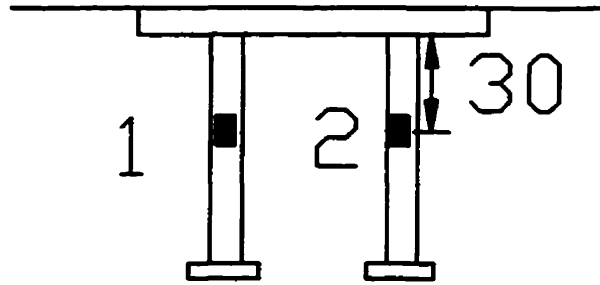


(a) Hysteresis response

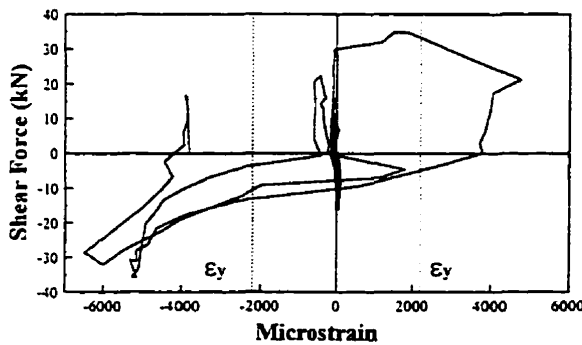


(b) Loading history

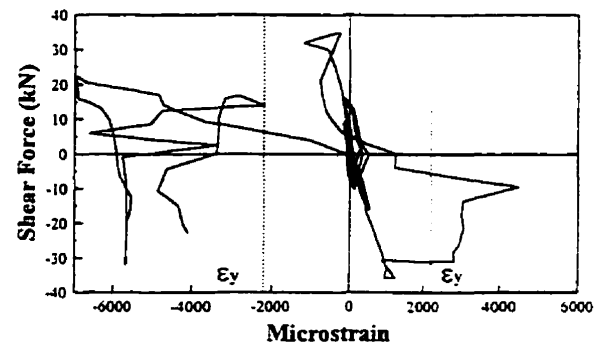
Figure 5.11 Hysteresis axial response and loading history of Specimen EST1



(a) Location of strain gauges

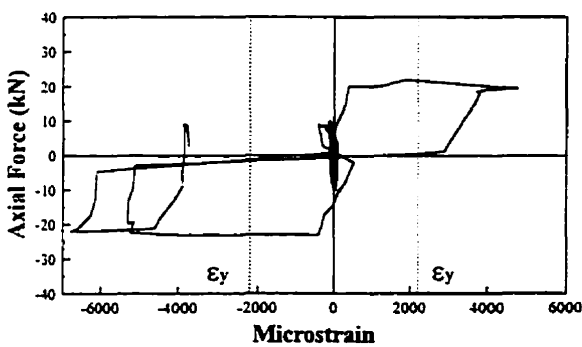


(b) Strain at Location 1 (shear response)

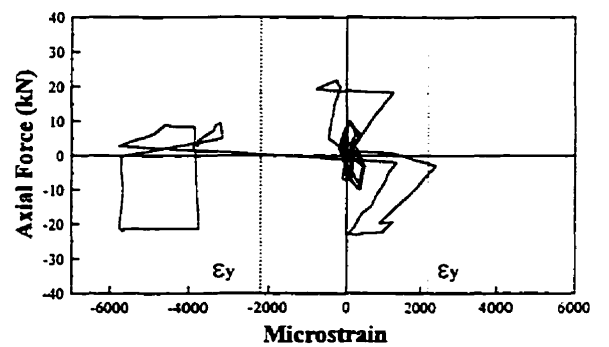


(c) Strain at Location 2 (shear response)

Figure 5.12 Measured strains in stud anchors for Specimen EST1(Shear response)

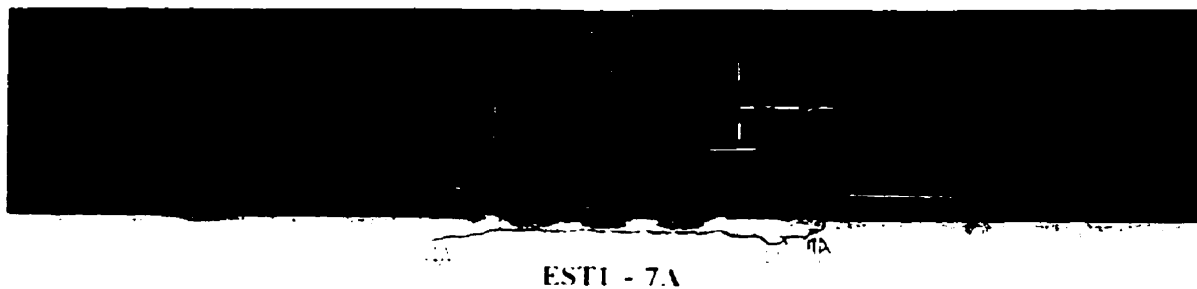


(b) Strain at Location 1 (axial response)

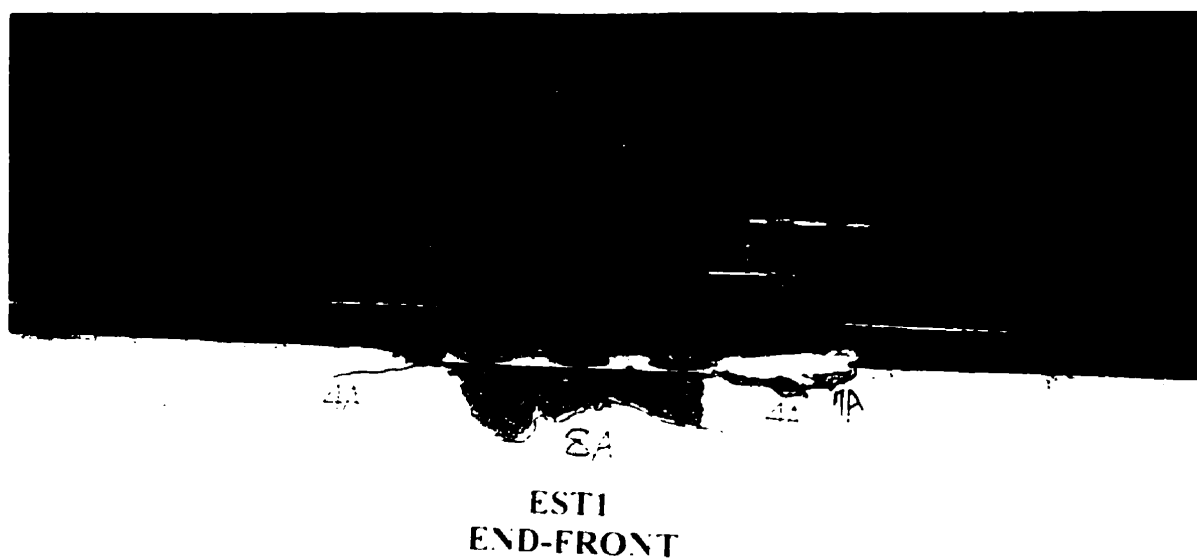


(c) Strain at Location 2 (axial response)

Figure 5.13 Measured strains in stud anchors for Specimen EST1(Axial response)



(a) Connection at first yield, Δ_v , $P_{\text{shear}}=35 \text{ kN}$, $P_{\text{axial}}=22 \text{ kN}$



(b) Connection after failure

Figure 5.14 Photographs of Specimen EST1

5.2.5 Specimen EST2

The loading history and the accompanying displacements for Specimen EST2 ($T/V=1.0$) are presented in Table 5.4. The load-deflection response and the loading histories for shear and axial loads are shown in Figures 5.15 and 5.16, respectively. The strains along the steel stud anchors for shear and axial loads are shown in Figures 5.17 and 5.18, respectively. The photographs of the specimen at yielding of the studs and at failure are shown in Figures 5.19(a) and 5.19(b), respectively. The following important behavioural aspects were noted from the observations during the experiment and from the analysis of the experimental results.

Cycles 1-3, $V=T=9$ kN The load-deflection curves remained linear elastic (Figures 5.15 and 5.16). The displacements were small (Table 5.4). No cracks appeared.

Cycles 4-6, $V=T=17$ kN The headed stud anchors at Location 1 yielded (Figures 5.17 and 5.18), where the measured horizontal and axial displacements were 0.3 mm and 0.5 mm, respectively. Crack appeared at the interface of the concrete and the steel plate, and extended toward the corner of the plate (Figure 5.19(a)). Due to yielding, the stiffness of connection decreased and the area under the hysteresis loops increased. The loops were narrow during negative loading (shear and compressive axial loads). At the end of this stage, the maximum interface crack width was 1.0 mm. The horizontal and axial displacements at the end of this stage were 0.4 mm and 0.8 mm, respectively (Table 5.4).

Cycle 07, $V=T=25$ kN This cycle was implemented using displacement control with deflection equal to $\Delta = 3\Delta_y = 1.9$ mm. The horizontal and axial displacements were 1.0 mm and 1.6 mm, respectively. The headed stud anchor yielded at Location 2, while the strain in the stud at Location 1 increased very significantly (Figures 5.17 and 5.18). The area under the hysteresis loops increased, showing the capability of connection to dissipate energy beyond the yielding of both studs. The concrete cover around the stud subjected to tensile load (Location 1) started to spall off and the width of the interface crack increased to 0.4 mm.

Cycle 8 $V=25$ kN $T=21$ kN As the connection was subjected to shear and axial loads simultaneously ($V=25$ kN and $T=21$ kN), the connection underwent large displacements and the stiffness of the connection decreased very significantly (Figures

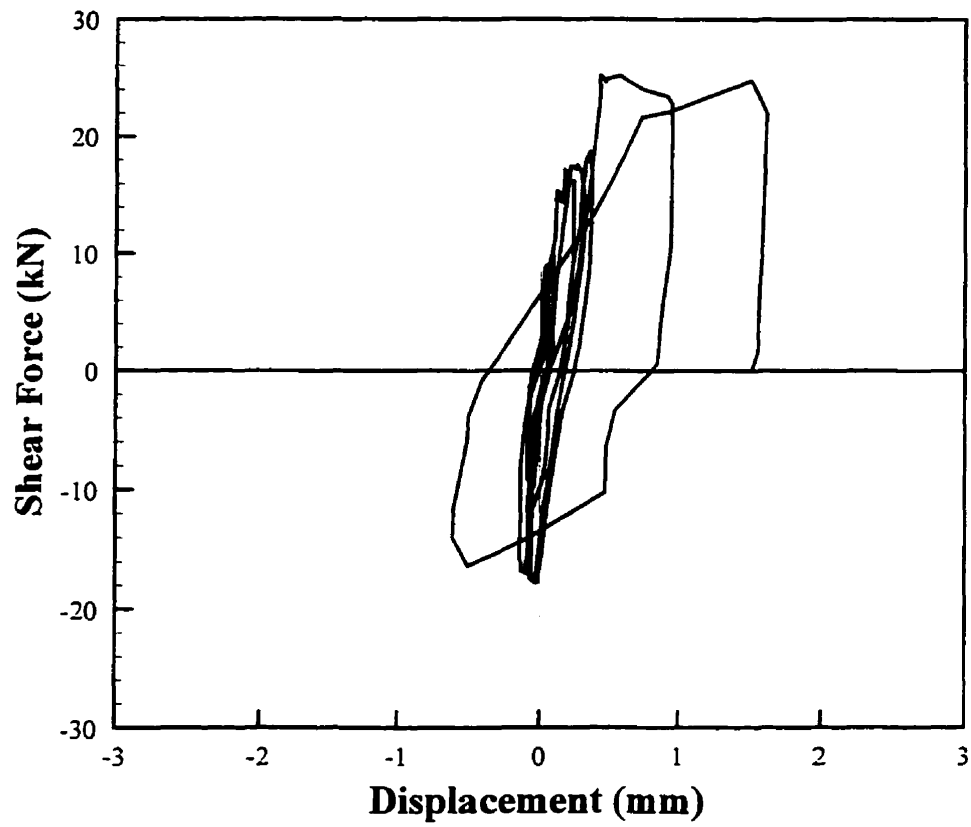
5.15 and 5.16). The area under the loops for both shear and axial responses increased significantly, indicating large amount of energy dissipation throughout this loading cycle. The concrete cover for the stud in Location 1 spalled off, and the concrete surrounding the steel plate broke away, while the stud subjected to tensile and shearing stresses (Location 1) got bent. When the load was applied in the negative direction, the connection could not resist any more load and the specimen failed due to fracture of the stud anchors. The ultimate horizontal and axial displacements were measured to be 1.6 mm and 2.3 mm, respectively, given displacement ductility ratio $\mu = 4.7$.

The maximum capacity of 25 kN was achieved for both shear and axial strengths. The final mode of failure was due to the complete deterioration of the concrete surrounding the steel plate and fracture of the stud anchors (Figure 5.19(b)). The failure mode was close to that for Specimen EST1. Based on the experimental results, under combined cyclic axial and shear loads ($T/V=1.0$), the strength of the connection reduced to about 54% and 78% of those for pure cyclic shear and pure cyclic axial strengths, respectively. This reveals that the effect of axial load on reduction of shear strength is more severe than the influence of shear on the axial strength. The hysteresis response showed large loops, which was mostly due to yielding of both studs.

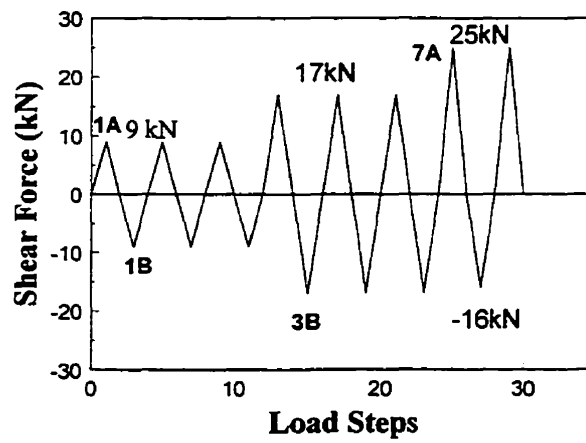
Table 5.4 Summary of loading history for Specimen EST2

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.1, -0.1	±9	+0.2, -0.01	±9
±1 Δy	4A - 6B	+0.4, -0.05	±17	+0.8, +0.2	±17
±2 Δy	7A - 7B	+1.0, -0.5	±25	+1.6, +1.0	±25
4.6 Δy , Failure	8A	+1.6	+25	+2.3	+21

* Maximum displacement at the peak load of the last cycle

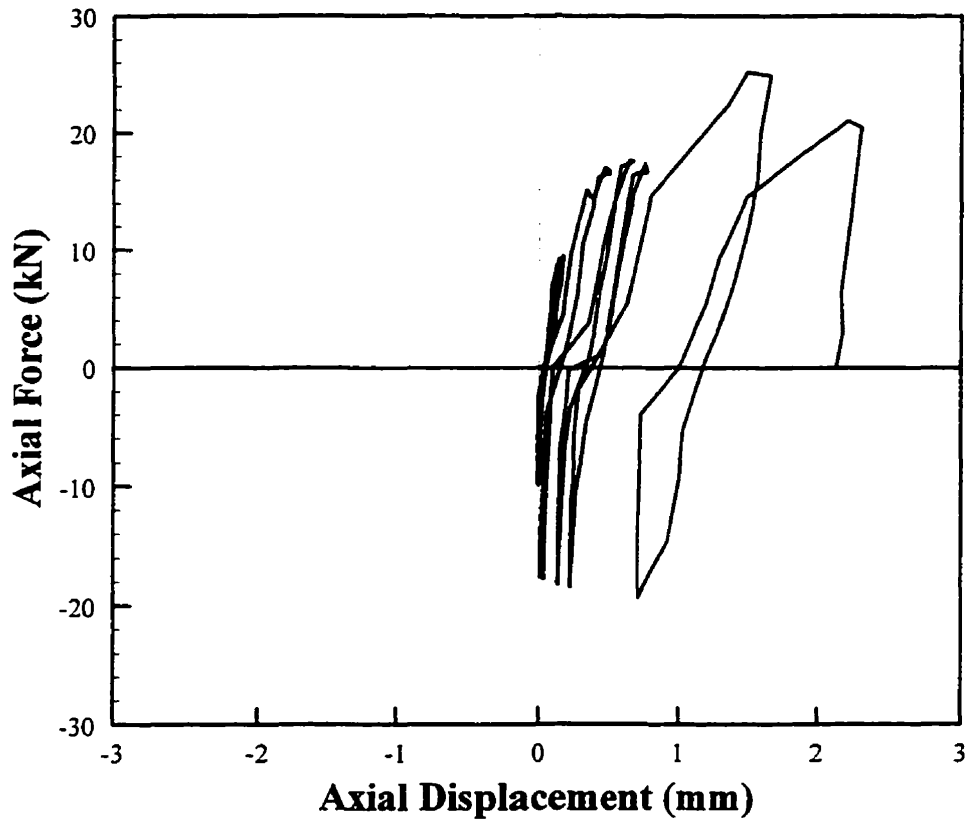


(a) Hysteresis response

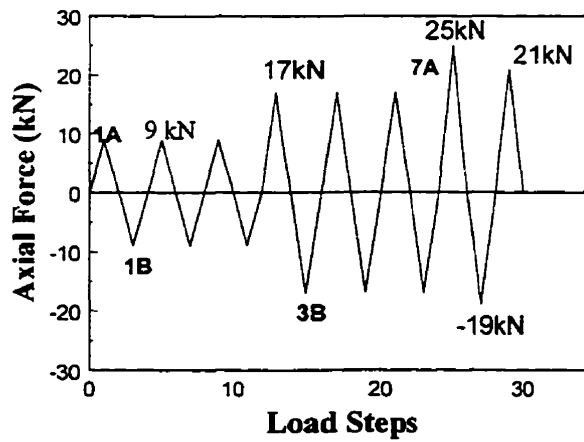


(b) Loading history

Figure 5.15 Hysteresis shear response and loading history of Specimen EST2

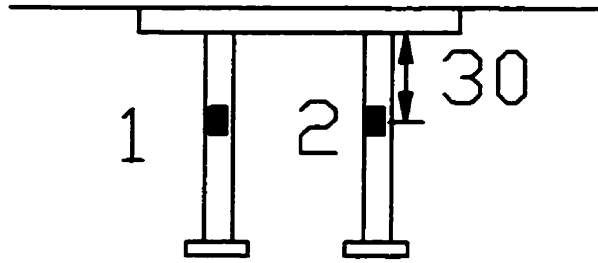


(a) Hysteresis response

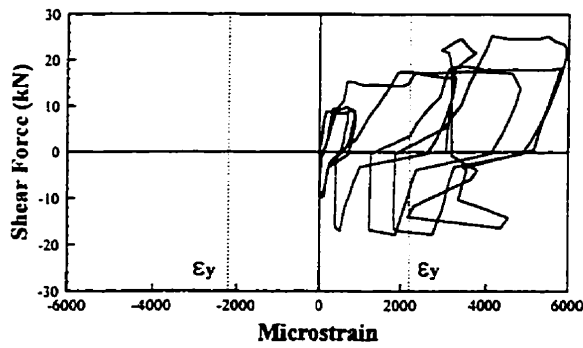


(b) Loading history

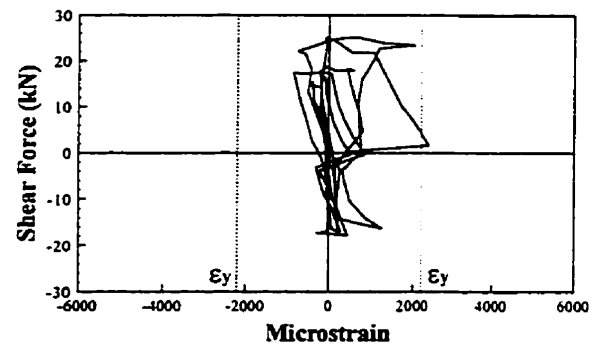
Figure 5.16 Hysteresis axial response and loading history of Specimen EST2



(a) Location of strain gauges

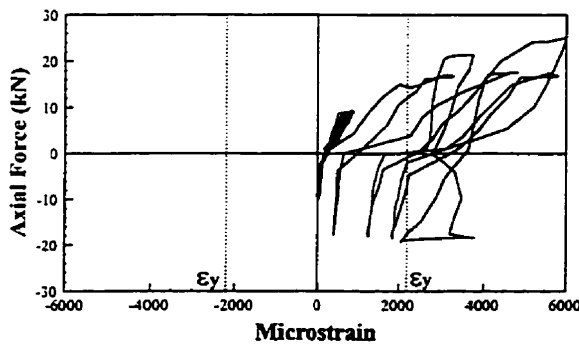


(b) Strain at Location 1 (shear response)

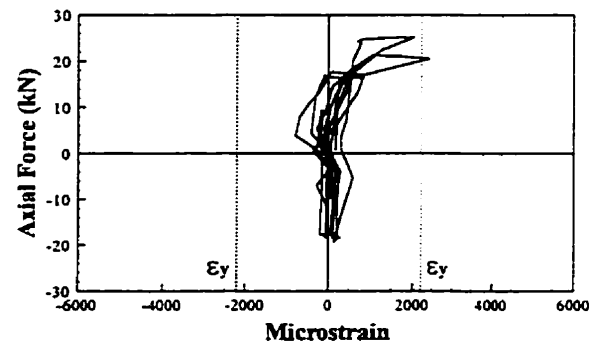


(c) Strain at Location 2 (shear response)

Figure 5.17 Measured strains in stud anchors for Specimen EST2 (Shear response)

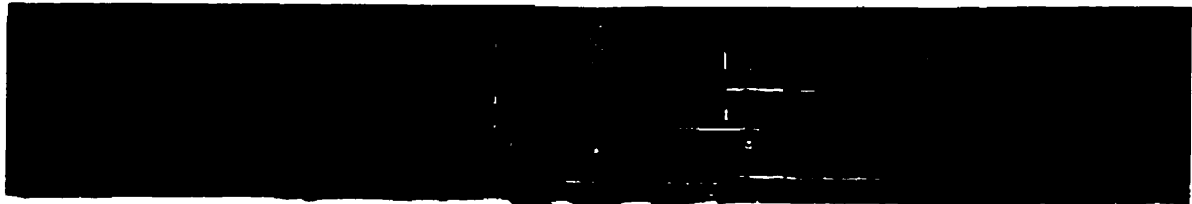


(b) Strain at Location 1 (axial response)



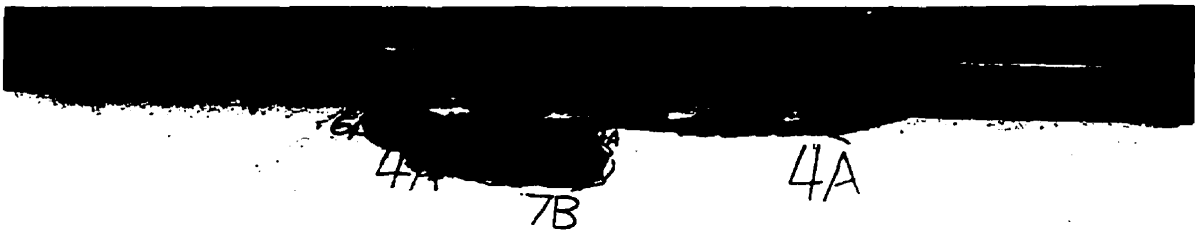
(c) Strain at Location 2 (axial response)

Figure 5.18 Measured strains in stud anchors for Specimen EST2 (Axial response)



EST2 - 4A

(a) Connection at first yield, Δ_y , $P_{\text{shear}} = P_{\text{axial}} = 17 \text{ kN}$



EST2
END-FRONT

(b) Connection after failure

Figure 5.19 Photographs of Specimen EST2

5.2.6 Specimen EST3

Table 5.5 summarizes the loading history and the accompanying displacements for Specimen EST3 ($T/V=0.3$). Figures 5.20 and 5.21 show the hysteresis response and the loading histories for shear and axial loading. The measured strain in the stud anchors for shear and axial forces are presented in Figures 5.22 and 5.23, respectively. Figures 5.24(a) and 5.25(b) show the photographs of the test specimen at yielding and failure, respectively. The following observation and results were noted during testing and analysis of the experimental results:

Cycles 1-3 $V=17\text{ kN}$ $T=5\text{ kN}$ Up to the end of Cycle 2, the response was linear elastic without any cracking. The first crack appeared at the concrete-steel plate interface resulting in a slight change of the slope of load-deflection curves (Figures 5.20 and 5.21).

Cycles 4-6 $V=26\text{ kN}$ $T=8\text{ kN}$ First yielding occurred along the stud anchor at Location 1 (Figures 5.22 and 5.23), which was mostly due to the shear forces. A diagonal crack appeared at the back of specimen and extended toward the ends of studs and the interface crack opened to about 0.3 mm (Figure 5.24(a)). The horizontal and axial displacements were both measured to be 0.4 mm. During the negative loading (shear and compressive axial forces) of the last cycle in this load stage, the headed stud at Location 2 yielded. The area under the hysteresis loops was not noticeable, indicating insignificant dissipation of energy .

Cycle 7, $V=40\text{ kN}$ $T=12\text{ kN}$ This cycle of loading was implemented using displacement control with the deflection equal to $2\Delta_y$. The horizontal and axial displacements were 0.7 mm and 0.9 mm, respectively. The area under the hysteresis loops increased significantly when compared with the area for previous load cycle. The concrete in front of the plate cracked and the width of interface crack was 0.8 mm.

Cycle 8, $V=40\text{ kN}$ $T=12\text{ kN}$ This stage of loading was implemented using deflection control with the deflection equal to $\Delta = 3\Delta_y$. The displacements were 1.1 mm and 1.4 mm for horizontal and axial displacements, respectively. The steel plate detached from the concrete and moved slightly in the horizontal direction. The inelastic strains at Location 1 stud anchor increased significantly (Figures 5.22 and 5.23). The slope of the load-deflection curves decreased when compared with previous cycle for the same load,

indicating reduction of the connection stiffness. The area under the hysteresis loops increased showing an increase in energy dissipation. The measured interface crack width was 1.1 mm.

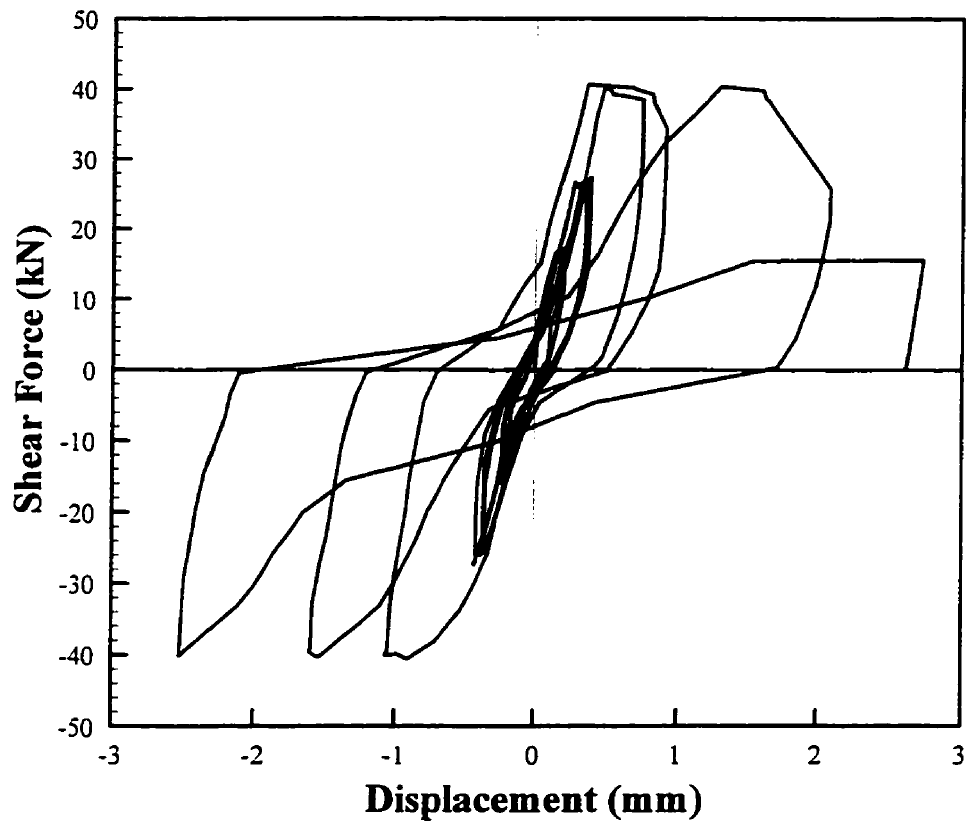
Cycles 9-10, $V=40$ kN $T=12$ kN This load cycle was implemented using deflection control and the deflection was equal to $4\Delta_y$. The stiffness of the connection decreased significantly (Figures 5.20 and 5.21). The concrete in front of the steel plate broke away resulting in a significant increase in the displacements. The shear force dropped to about 26 kN. The compressive strain in the stud at Location 2 exceeded its yield value (Figures 5.22 (b) and 5.23 (b)). The maximum horizontal and axial displacements at this stage were 1.6 mm and 1.7 mm, respectively ($\Delta=4\Delta_y=2.3$ mm). The area under the hysteresis loops increased indicating the capability of the connection to dissipate energy. The hysteresis responses revealed stable loops (Figures 5.20 and 5.21). During Cycle 10, as the 15 kN shear force was applied, the stiffness and strength of the connection reduced very significantly. Both stud anchors had necked and fractured resulting in the failure of the connection (Figure 5.24(b)). The connection underwent very large displacement at small incremental loads during the last cycle (unstable loops), therefore the displacement during the previous cycle (Cycle 9) was used to calculate the connection ductility. The displacement ductility ratio at this loads stage was of $\mu = 2.3/0.6 = 4$.

The maximum shear and axial strengths achieved were 40 kN and 12 kN, respectively. The failure mode was due to necking and fracture of the headed stud anchors and deterioration of the concrete surrounding the steel plate (Figure 5.24(b)). The mode of failure was similar to that for Specimen ES. The hysteresis response showed large loops during the last cycles, which is an indicator of energy dissipation for the connection after general yielding of both stud anchors. The experimental results showed that if less than 30% of shear force is applied simultaneously in the axial direction in a cyclic manner, the strength of connection reduces to about 87% of that for pure cyclic shear.

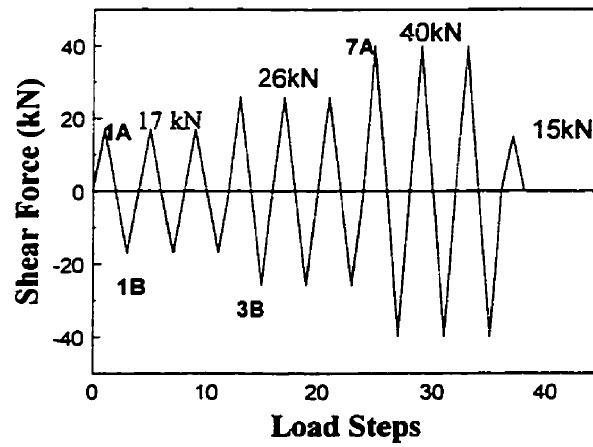
Table 5.5 Summary of loading history for Specimen EST3

Cycle	Load Stages	Horiz. Disp. (mm)*	Shear Force (kN)	Axial Disp. (mm)*	Axial Force (kN)
Elastic	1A - 3B	+0.2 , -0.2	±17	+0.2 , -0.1	±5
±1 Δ y	4A - 6B	+0.5 , -0.4	±26	+0.4 , -0.05	±8
±2 Δ y	7A - 7B	+0.7 , -1.0	±40	+0.9 , +0.2	±12
±3 Δ y	8A - 8B	+1.1 , -1.6	±40	+1.4 , +0.4	±12
±4 Δ y	9A - 9B	+1.6 , -2.5	±40	+1.7 , +0.7	±12
Failure	10A	+2.7	+15	-	-

* Maximum displacement at the peak load of the last cycle

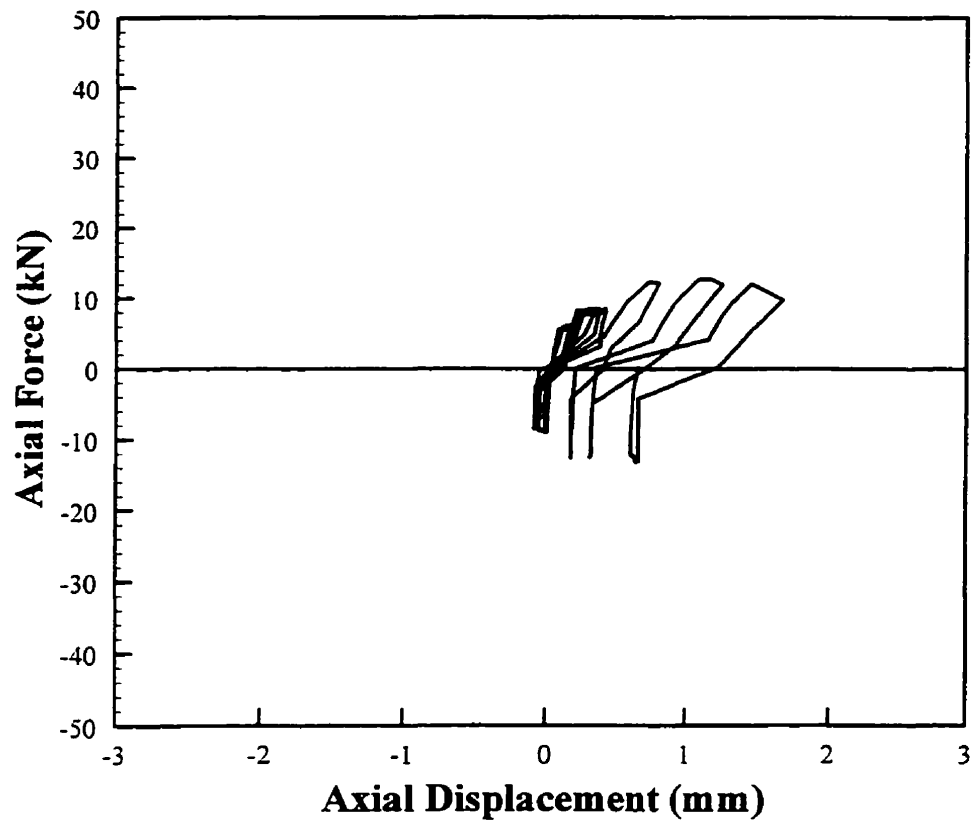


(a) Hysteresis response

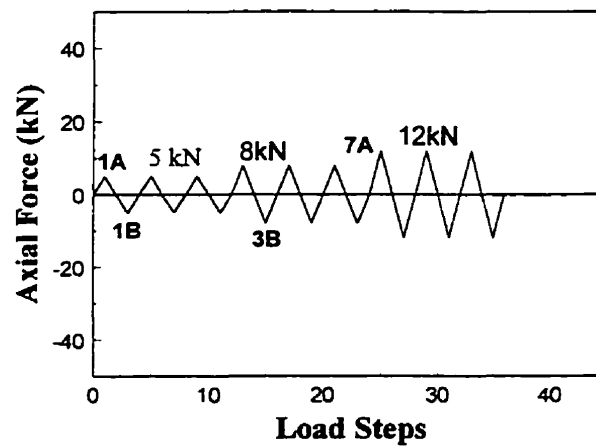


(b) Loading history

Figure 5.20 Hysteresis shear response and loading history of Specimen EST3

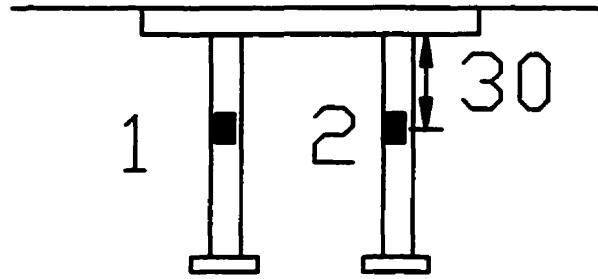


(a) Hysteresis response

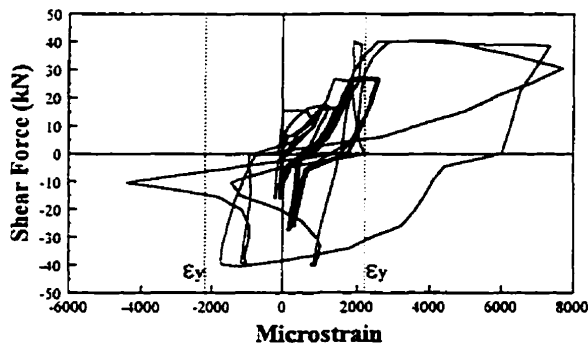


(b) Loading history

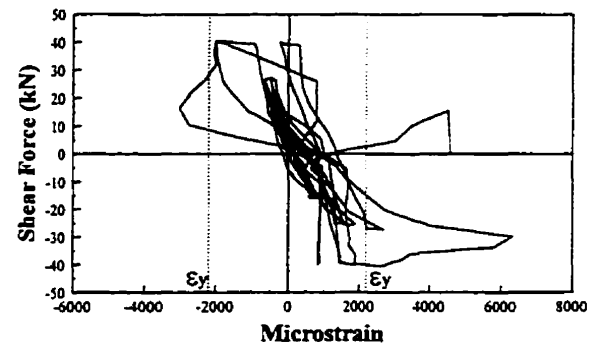
Figure 5.21 Hysteresis axial response and loading history of Specimen EST3



(a) Location of strain gauges

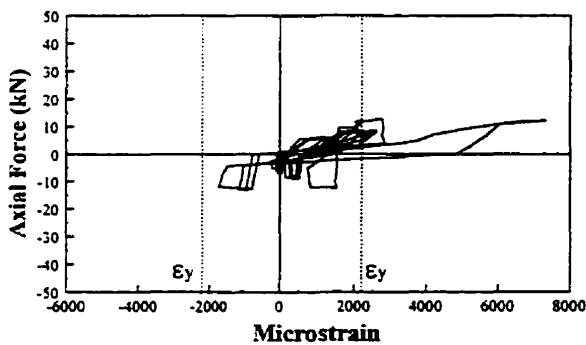


(b) Strain at Location 1 (shear response)

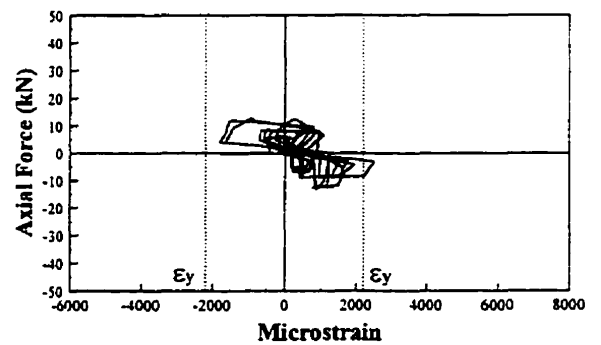


(c) Strain at Location 2 (shear response)

Figure 5.22 Measured strains in stud anchors for Specimen EST3 (Shear response)



(b) Strain at Location 1 (axial response)

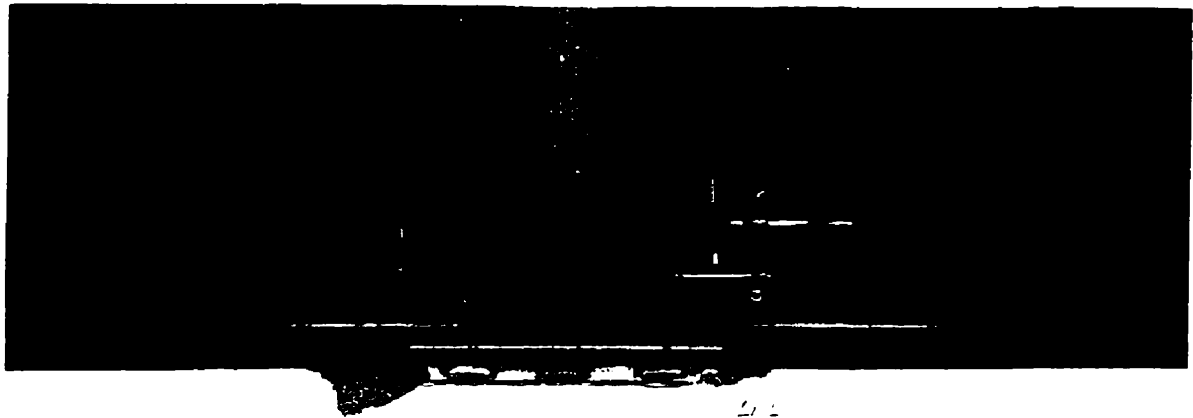


(c) Strain at Location 2 (axial response)

Figure 5.23 Measured strains in stud anchors for Specimen EST3 (Axial response)



(a) Connection at first yield, Δ_y , $P_{\text{shear}}=26 \text{ kN}$, $P_{\text{axial}}=8 \text{ kN}$



EST3
END-FRONT

(b) Connection after failure

Figure 5.24 Photographs of Specimen EST3

5.2.7 Interaction Diagram for Series E

In order to design safe precast structural systems using the headed stud connection, one should know the strength of the connections under the various combinations of shear and axial forces induced during an earthquake. To gain this knowledge, a detailed investigation was carried out to develop the interaction diagram and to determine the equation for combined shear and axial strength of the connection.

The behaviour information and the ultimate capacities of the specimens in Series E subjected to different combination of cyclic shear and axial loads were examined in the previous sections (Sections 5.2.2 to 5.2.6). Based on the results obtained, and as summarized in Table 5.6, the maximum cyclic shear strength ratios, expressed as V/V_0 , are plotted versus the cyclic axial strength ratios, T/T_0 , in Figure 5.25. An interaction equation was fitted to the experimental results using the best fit program, giving the following interaction equation:

$$\left(\frac{V}{V_0}\right) + \left(\frac{T}{T_0}\right)^{3.5} \leq 1.0 \quad (5.1)$$

where, V is the applied shear force, V_0 is the maximum force that connection can resist in pure cyclic shear, T is the applied axial force, and T_0 is the maximum force that connection can resist in pure axial. The material resistance factors must be included for the equation for design application (see Chapter 7).

It is noted from Figure 5.25 that the cyclic shearing capacity of the connection decreases when cyclic axial force is applied simultaneously, because the axial forces induce stresses in the concrete surrounding the headed stud anchors and the steel plate, which result in a reduction of the strength when the shear forces are applied simultaneously. However, if less than 30% of shear force is applied simultaneously in the axial direction, the shear strength of connection dose not reduce significantly. Besides, the cracking load and the shear stiffness decrease as a result of the application of the axial force. Similarly, the cyclic axial strength decreases when a cyclic shear force is applied.

Table 5.6 Summary of interaction test results for Series E

Test type	Ratio of axial to shear (T/V)	Ultimate strength (kN)		Ratios	
		Shear (kN)	Axial (kN)	V/V _o	T/T _o
ES	-	46	-	1.00	0.0
EST1	0.7	35	22	0.78	0.69
EST2	1.0	25	25	0.56	0.78
EST3	0.3	40	12	0.90	0.40
ET	-	-	32	0.0	1.0

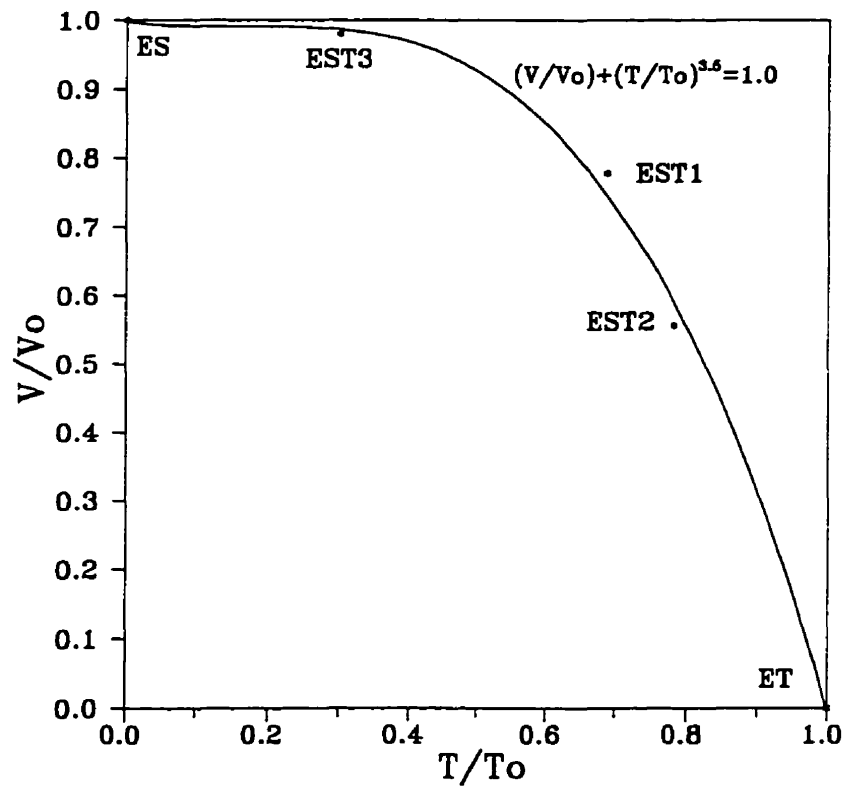


Figure 5.25 Normalized interaction diagram for Series E

Chapter 6

Comparison and Discussion of Experimental Results

6.1 Introduction

This chapter presents a detailed discussion and comparison of the responses of different types of connections, subjected to monotonically increasing and reversed cyclic loads. The complete behavioural responses were used to develop relevant characteristics related to each of thirty-one specimens, and the differences between the connection responses subjected to monotonically increasing and cyclic loads, ultimate strengths, displacement ductilities, failure modes and energy dissipation capacities. The discussion and response comparisons are arranged in two categories; 1) comparison of responses of different connection type (Series A through Series E), and 2) comparison between identical specimens subjected to different loading types (monotonically increasing and reversed cyclic, termed monotonic and cyclic loadings throughout this chapter) and different combinations of cyclic shear and cyclic axial loads (different ratio of shear to axial forces, T/V ratios).

6.2 Response Comparison of Each Connection Type (Series A to Series E)

In this section, the monotonic and cyclic responses of each connection type are discussed and compared along with the cyclic responses. Connection performances are compared based on their behavioural responses and the ability to dissipate energy through inelastic behaviour.

6.2.1 Monotonically Increasing Load Response

Figure 6.1 compares the shear force - horizontal displacement curves for all five type of connections tested under monotonically increasing loads until failure. The cracking, yielding and the ultimate strengths of the connections along with their failure

modes are presented in Table 6.1. Before initial cracking, all of the connections showed high stiffness and the load-deflection curves were linear (Figure 6.1). The initial stiffness, for Specimen BMS was the highest compared with the other specimens which was due to the provision of the hook. As can be seen from Figure 6.1 and Table 6.1, the cracking load for Specimen BMS with hooks at the bar ends was the highest, while the Specimen EMS reinforced with the headed studs showed the lowest cracking load. The reinforcement anchors in the connections yielded at loads of 110, 165, 125, 205, and 45 kN for Specimens AMS, BMS, CMS, DMS, and EMS, respectively (Table 6.1). Specimen DMS attained the highest yielding load compared with the other connection types. This was due to provision of an extra reinforcing steel anchor oriented at 90°, which provided additional strength to the connection and in turn increased the force required to yield any of the anchor bars. The Specimen BMS showed the larger yielding strength after Specimen DMS, which was attributed to 90° hook provided at the end of the bars. The maximum crack widths at the yield stage for Specimens AMS, BMS, CMS, and DMS were 0.8, 1.0, 1.0 and 1.0 mm, respectively. Since the design of a connection is based on the yield strength of the reinforcing bar anchor, the maximum crack width at the stage of yielding is important as the aesthetic appearance of connection is very important in a structure.

All of the specimens were able to sustain load even after yielding of a considerable length of the reinforcing anchor, and displacement ductility ratios ($\mu = \Delta_{ultimate} / \Delta_y$) of 1.7, 3.8, 3.7, 3.1 and 7.8 were achieved for Specimens AMS, BMS, CMS, DMS and EMS, respectively. Specimen BMS showed an improved response compared to Specimen AMS (Figure 6.1) because of the provision of the standard hook. The largest experimental ultimate strength of 242 kN was attained for this specimen (Table 6.1 and Figure 6.1). Specimen DMS displayed an improved behavioural response compared with the Specimen AMS (see Figure 6.1) because of the provision of an additional orthogonal reinforcing bar in the middle of the connection. This orthogonal reinforcing anchor prevents the out of plane movement of the connection, thus preventing the spalling of concrete as was the case for Specimen AMS. This spalling caused reduction in the ultimate strength of the specimen.

The failure mode for all of the connections subjected to monotonic loading was due to failure of the concrete, except for Specimen EMS which was due to exhaustion of the shear strength of the studs (shear fracture failure). The details of failure modes for the specimens are presented in Table 6.1.

In terms of energy absorption capacity, represented by the area under the load-deflection curve, the best energy absorption characteristics are shown by Specimen DMS, which is a consequence of the extra orthogonal reinforcing bar. The orthogonal anchor caused very gradually decrease of connection stiffness due to a uniform crack distribution across the panel. The Specimen BMS showed great energy absorption, close to that for the Specimen DMS, due to provision of the 90° standard hook. The Specimen CMS, reinforced with two orthogonal bars, also showed good energy absorption capacity. Specimen EMS had the smallest area under the load-deflection curve and thus the smallest energy absorption capacity.

Connection Types	Cracking load (kN)	Yielding load (kN)	Ultimate strength (kN)	Failure modes
AMS	85	110	187	Splitting-spalling along tensile leg
BMS	95	165	242	Splitting-spalling along tensile leg plus crushing-spalling in front of 90° hook
CMS	70	125	152	Splitting - crushing failure along tensile leg
DMS	65	205	226	Failure along diagonal crack
EMS	29	45	53	Shear fracture of headed studs

Table 6.1 Comparison of connection response parameters subjected to monotonic load

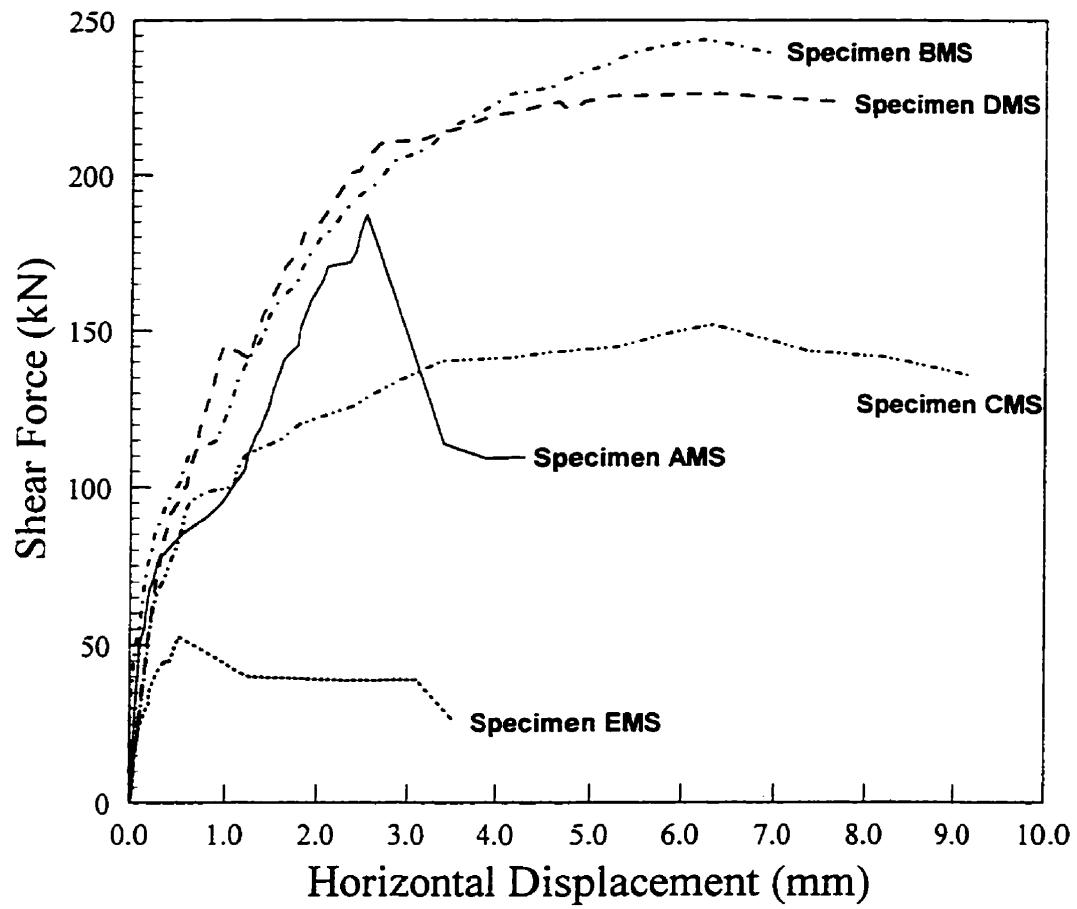


Figure 6.1 Comparison of shear force-horizontal displacement responses of connections subjected to monotonic loading

6.2.2 Reversed Cyclic Load Response

Comparison of the responses of the connections, with different details, subjected to pure reversed cyclic shear and axial loads are made. The connection performances were evaluated on the basis of the hysteresis responses, ultimate shear and axial strengths, strength degradation through the various cycles, mode of failure, displacement ductility ratio and the energy dissipation characteristics.

6.2.2.1 Pure Reversed Cyclic Shear Responses

Hysteresis response The shear force-displacement hysteresis loops exhibited similar characteristic behaviour for all type of connections. The hysteresis loops show the degradation of the stiffness and the load-carrying capacity during the repeated cycles, which is the characteristic of connection in precast concrete structures due to cracking of the concrete and yielding of the reinforcing steel anchors. All of the connections experienced some degree of pinching depending upon the connection details. Pinching is characterized by the narrow width of the hysteresis loops near the mid-cycles. Pinching in precast connection is caused mainly by: 1) combination of dowel action and bond stresses resulting in the shear deformation of the concrete, 2) slippage of the steel anchors, 3) closing of wide diagonal and splitting cracks. Most of the pinching in the hysteresis loops was accredited to Items 2 and 3. All connections displayed linear response during the earlier stages of loading.

The cracking , yielding and ultimate strengths are compared in Table 6.2. The Specimen BS achieved the highest cracking load, while Specimen ES reinforced with the headed studs exhibited the lowest cracking strength. The Specimen AS displayed a cracking load close to that for Specimen BS. Since the hook in the Specimen BS did not play an important role up to cracking, the responses of the Specimens BS and AS in the elastic range up to cracking were similar and the cracking strengths of the two specimens were close to each other. Specimen CS and DS had the same cracking load. In terms of yielding load capacities, Specimens AS, BS and DS exhibited the same yielding load value. This was related to the load transfer mechanism at the yielding stage, with most of the applied load being carried by the inclined legs. Since the inclined legs

were used in all of these connection (Specimens AS, BS, DS), the yielding loads were similar.

Strength degradation and ultimate strength The shear load-horizontal displacement envelopes for the various connections are shown in Figure 6.2. The Specimen AS exhibited more strength degradation compared to the other specimens, while Specimens BS, CS and DS maintained their load-carrying capacity after the maximum load was attained. This was related to the fact that the hooks in Specimen BS helped to resist the load after the exhaustion of the bond resistance along the straight part of the anchor legs. In Specimen DS, the contribution of the orthogonal bar at the middle of connection helped with better performance under reversed cyclic loads. For Specimen CS, the dowel resistance and partial confinement of the concrete between the legs after complete splitting along the bar legs (maximum load) assisted with the sustaining of the post-peak resistance. The envelope response for Specimen DS was stiffer compared to that for Specimen AS (Figure 6.2), which is due to the effect of the additional orthogonal bar. Specimen DS also showed larger envelope and displacement (Figure 6.2). The curves in Figure 6.2 and Table 6.2 show that the Specimen BS attained the greatest strength (200 kN) in comparison with the other specimens. Comparing the ultimate strengths for Specimens AS and BS indicates that the provision of standard hook caused considerable improvement in the connection strength. The Specimens AS, CS and DS achieved ultimate strengths of 150, 125 and 180 kN, respectively. Comparison of Specimen DS with Specimens AS and CS shows that with an additional orthogonal bar, the Specimen DS results in an increase in stiffness, yield and ultimate applied cyclic shear, post-peak load sustainability, and the displacement at yield and ultimate load (Figure 6.2)

Mode of failure The modes of failure for the various specimens are listed in Table 6.2, and described in Chapters 3, 4 and 5. All of these connections experienced some concrete crushing around the steel angle, or the steel plate. The Specimen AS failed mainly by longitudinal splitting along the anchor leg [see Figure 3.8(b)]. The connection BS failed due to the crushing of the concrete in front of 90° standard hook [see Figure 3.39(b)].

This was the result of being to resist all of the tensile load in front of the hook in the tensile zone after concrete splitting along the straight part of the anchor leg. The mode of failure for the Specimen CS was longitudinal splitting along both orthogonal anchor legs and crushing of the concrete between the two legs [see Figure 4.8(b)], resulting from the bearing stresses induced in the bar by the shearing forces (dowel action). Specimen DS failed along the diagonal crack which had formed crossing the end of the three anchor bars [see Figure 4.39(b)]. This was due to the additional orthogonal anchor provided at the middle of the connection causing the connection to be stiff. In fact, the failure occurred on the "cone surface area", which was a consequence of integrity of the concrete surrounding the two inclined and one orthogonal legs in the connection causing strong region around the three bars. The Specimen ES failed due to the shear fracture of the steel studs as a consequence of transferring direct shearing stresses to the studs.

Energy dissipation and ductility Figure 6.3 compares the cumulative energy dissipated (summation of the energy dissipated in each cycle) during the test versus the ductility level for pure cyclic shear loading of the connections. The energy dissipated by each specimen was computed as the area enclosed within the hysteresis loop for the particular cycle. As can be seen from Figure 6.3, the energy dissipated and the displacement ductility were the greatest in the Specimen DS than in the other specimens. The curves in Figure 6.3 confirm the observation that pinching was more severe in Specimens CS and BS. Pinching decreases the area enclosed within the hysteresis loops, thus reducing the amount of energy dissipated. The performance of Specimens AS through DS were similar up to the yielding stage (Figure 6.3). After yielding, the Specimens AS and DS preformed better than Specimens BS and CS. The improved performance of Specimen DS was attributed to the orthogonal anchor provided at the middle of the connection. In Specimen BS, after complete splitting crack along the straight part of the anchor, the hook caused the connection to be more stiff and to deteriorate at a faster rate. Sudden failure due to spalling of the concrete in front of the hook resulted in an out of plane movement. These phenomena increased the pinching and influenced the connection performance, decreasing the energy dissipation and displacement ductility. Specimen ES reinforced

with headed studs had the smallest area under the loops, and therefore the smallest energy dissipation (Figure 6.3). The fact is that the amount of energy dissipated was dependant upon the strength and the displacement, since the ultimate strength and displacement for Specimen ES were the lowest, resulting in the lowest energy dissipation.

Connection Types	Cracking load (kN)	Yielding load (kN)	Ultimate strength (kN)	Failure modes
AS	77	150	150	Failure along the tensile leg and along the diagonal cracks
BS	82	150	200	Splitting-spalling along the tensile leg plus crushing-spalling in front of the 90° hook
CS	65	105	125	Splitting failure along the tensile leg and crushing of concrete between two legs
DS	65	150	180	Failure along the diagonal crack formed at the ends of the bar legs
ES	15	25	46	Shear fracture of headed studs and crushing of concrete around plate

Table 6.2 Comparison of response parameters of Specimens AS through ES
(Specimens subjected to pure cyclic shear load)

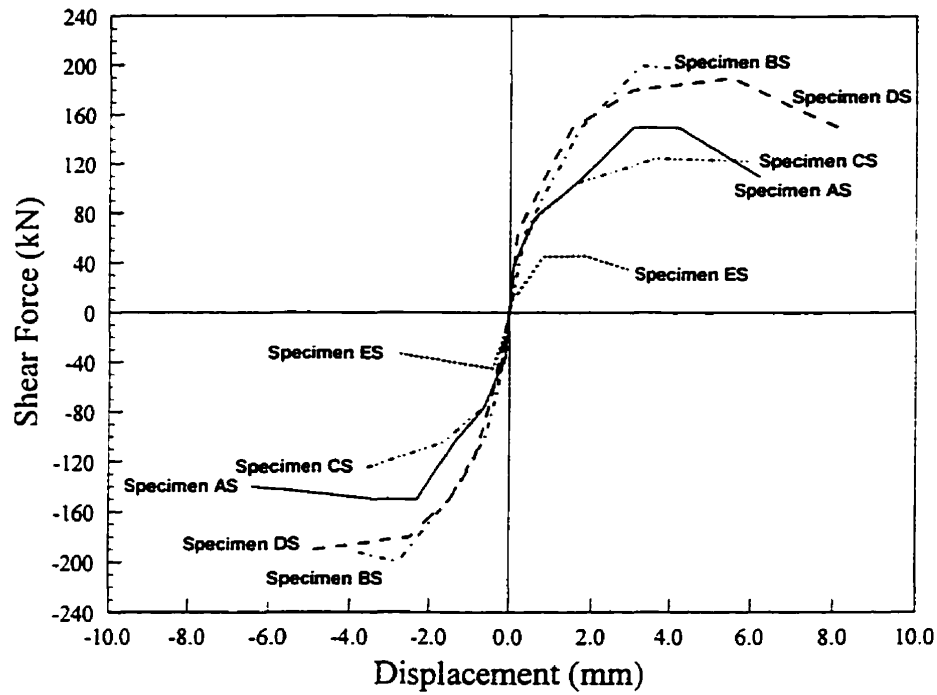


Figure 6.2 Envelopes of hysteresis responses of Specimens AS through ES

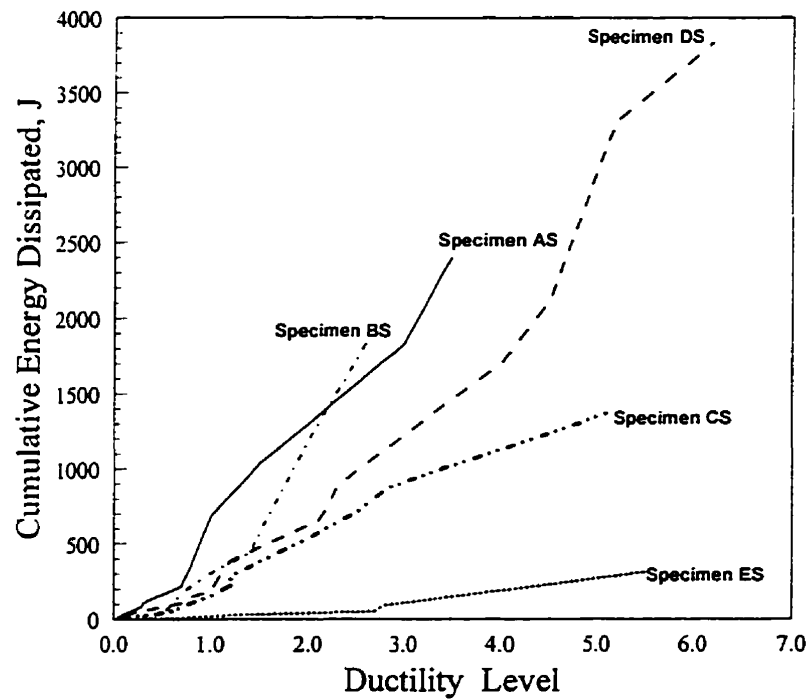


Figure 6.3 Cumulative hysteresis energy dissipation of Specimens AS through ES

6.2.2.2 Pure Reversed Cyclic Axial Responses

Hysteresis response The nature of the hysteresis response of the Specimens AT through ET (specimen subjected to pure cyclic axial loads) were similar, typically with the connection responding in a tension-compression mode of behaviour. The hysteresis response showed large loops in the positive loading direction (shear and tensile loads), which was the development of the yielding along the reinforcing steel anchors, accompanied by extensive diagonal and splitting cracks. However, the small loops in the negative direction were a consequence of the imperfect closing of the cracks and small deformability of the concrete subjected to compressive stresses. The connection stiffness and load-resistance were observed to decrease throughout the repeating cycles due to crack opening, crack propagation and yielding of steel anchor. The response during the first cycles were basically linear for all connection types. The cracking, yielding and ultimate strengths along with the final mode of failure are listed in Table 6.3 for Specimens AT through ET. Table 6.3 shows that Specimens BT and CT had the highest cracking load (70 kN) and Specimen ET exhibited the lowest strength at cracking. The cracking strengths for Specimens AT and DT were the same (65 kN). This may be explained, as the additional orthogonal anchor in type D connection was not effective up to the cracking load. Having larger cracking strength for Specimen BT compared with that for AT indicates that provision of a standard hook is useful even at early stages of loading. The degradation of stiffness after cracking for Specimen AT was more pronounced than for the other specimens. This was due to the very small additional cracks which appeared at the concrete-steel angle interface. The provision of hook and additional orthogonal reinforcing anchor in Specimens BT and DT, respectively, prevented the reduction of the stiffness after cracking, as compared with that for Specimen AT. The yielding load for the Specimen DT was the highest due to additional orthogonal bar. The additional anchor in the Specimen DT caused the axial load to be distributed between three legs which in turn reduced the stress in each leg compared with stresses in the Specimen AS. This led to a higher load to yield these anchors. The Specimen ET with the stud anchors exhibited the smallest yield load because of small axial cross sectional area of the stud resulting in a lower yield strength.

Strength degradation and ultimate strength The axial load-axial displacement hysteresis envelope curves for the various connections are presented in Figure 6.4. The Specimens AT and CT show more strength degradation, while the Specimens DT and BT were able to maintain their strengths beyond the maximum load. This was because of the 90° standard hooks and the additional orthogonal reinforcing anchor provided in the Specimens BT and DT, respectively. In general, the nature of hysteresis response envelope for Specimen DT was stiffer when compared with Specimen AT, basically because of the additional orthogonal anchor provided in Specimen DT, which reduced the rate of deterioration, and consequently it led to a stiffer response. Providing standard hooks at the end of the reinforcing bars in the Specimen BT, resulted in an improvement in the performance of the connection under reversed cyclic axial forces (see Figure 6.4).

The Specimen DT attained the highest ultimate strength (180 kN) (Figure 6.4 and Table 6.3). The specimens BT and CT indicated the same ultimate strength, however, Specimen BT showed a better overall performance (see Figure 6.4). The lowest ultimate strength was exhibited for Specimen ET, reinforced with headed studs. This was a consequence of small stud length which was pulled out under tensile axial load. Comparing the hysteresis envelopes for the Specimen DT with those for Specimens AT and CT, shows that the provision of an additional orthogonal anchor in Specimen DT, increases its stiffness, yielding and the ultimate axial load and the post-peak load sustainability, and the displacement at ultimate load (see Figure 6.4 and Table 6.3). The small ultimate strength for Specimen AT was attributed to the extensive spalling of the concrete along the bar anchor close to the loading plate.

Modes of failures The modes of the failure for the specimens subjected to pure reversed cyclic axial loads are listed in Table 6.3, and were described in Chapters 3, 4 and 5. The failure for all specimens started primarily with disintegration of the concrete surrounding the steel angle, or plate. The Specimen AT failed mainly by splitting and spalling of the concrete along both bar legs [see Figure 3.12(b)], which was the cause of low stiffness as shown in Figure 6.4. Near the ultimate load, there was extensive spalling of the concrete surrounding the anchor close to the steel angle, which led to further extension

of the splitting cracks along the reinforcing anchor legs, which must be avoided to prevent failure at relatively small displacements. The use of fabric mesh on the both face of the specimen will confine the concrete surrounding the anchors, and prevent the spalling-splitting type of failure. This will improve the overall hysteresis response of the type A connection. In the Specimen BT splitting bond failure along the straight part of the anchor resulted in the development of low bond stresses along that part, therefore the standard hooks sustained most of the applied load. Under cyclic loading, the concrete in front of the hooks crushed and spalled off, resulting in the final failure of the connection [see Figure 3.43 (b)]. The Specimen CT failed due to the complete destruction of the concrete surrounding both legs. This was a consequence of the direct transfer of the axial load to the connection through bond stresses at the concrete-steel anchor interface. Since two legs were close to each other, the concrete between them crushed under reversed loading, while the concrete surrounding the outer-face of the legs exhibited splitting cracks. Failure of the Specimen DT occurred when the concrete along the continuous diagonal crack crossing the ends of the anchors failed completely. This mode of failure was the result of the induced stress distributions to the concrete surrounding the bar anchors. Hence, the concrete helps to develop the strength of the anchor groups fully. Thus the equation for calculation of the strength of a group of studs "tensile-cone" (CPCI Design Manual) can be modified to be used for computing the strength of this type of connection, However this computational strength must be checked with the strength calculated using the truss analogy. The Specimen ET failed due to pull out of the stud anchors. It occurred when due to the reversing load, the strength of the concrete on the surface of a cylinder from the bearing edge of the stud to the concrete surface got exhausted. The strength of the specimen subjected to cyclic axial load can be calculated based on the observed mechanism of failure. The proposed equation can be expressed in terms of the concrete cylinder surface area, P times L , and tensile strength of the concrete (f_c) given:

$$T_{ultimate} = nPLf_c' \quad (6.1)$$

where P , L and n are the perimeter of the stud head, its embedment length and the

number of studs in the connection, respectively. However, this equation must be checked against those recommended in the Metric Design Manual, with the smallest value governing the ultimate strength. A reduction factor to account for the seismic loading should be applied.

Energy dissipation and ductility Figure 6.5 compares the cumulative energy dissipated versus the ductility for each connection type subjected to pure reversed cyclic axial (tension-compression) loads for the Specimens AT through ET. The cumulative energy dissipated and the displacement ductility ratio for Specimen BT were the largest in comparison with the other specimens, as shown in Figure 6.5. This was related to the 90° standard hooks provided at the end of anchor bars. In fact after splitting of the concrete along the straight part of the anchors, almost all of the applied axial load was transferred to the hooks as bearing forces. This developed compressive stress in the concrete located in front of the hooks. Under reversed cyclic axial loads, the concrete at these region preformed well, providing a good overall response for the connection. Semi-confinement of the concrete surrounding the hooks will improve all aspects of the connection behaviour. As noted in Figure 6.5, the dissipated energy was greater in the Specimen CT than that in the Specimen DT, although the displacement ductility ratio for the Specimen DT ($\mu=5.5$) was larger than that for the Specimen CT ($\mu=4.7$). This was due to the narrow loops exhibited in the hysteresis response of the Specimen DT, accounting for the reduction in the cumulative energy dissipation. The contribution of the orthogonal reinforcing bar in the connection resulted in a stiffer response under cyclic axial loading, specially before significant opening of the diagonal cracks. However, up to a displacement ductility ratio of 2, the performances of the Specimens DT and CT were similar. Specimen AS revealed small dissipated energy and displacement ductility ratio (see Figure 6.5), which was attributed to the extension of the concrete splitting along the anchor legs and extensive concrete spalling close to the connection steel angle. The Specimen ET showed the smallest capability to dissipate energy, although the displacement ductility ratio was large, as shown in Figure 6.5. The smaller energy dissipation in Specimen ET was associated mostly with its ultimate strength which was the smallest in comparison

with the other specimens.

Connection Types	Cracking load (kN)	Yielding load (kN)	Ultimate strength (kN)	Failure modes
AT	65	95	106	Splitting-spalling along both legs
BT	70	100	150	Splitting-spalling along both legs plus crushing-spalling in front of the 90° hooks
CT	70	115	150	Complete deterioration of concrete (cracking-crushing) surrounding the reinforcing anchor legs
DT	65	135	180	Failure along the continuous diagonal crack crossing the ends of all three bar legs
ET	23	28	32	Pull out of the stud anchors

Table 6.3 Comparison of response parameters of Specimens AT through ET
(specimens subjected to pure cyclic axial load)

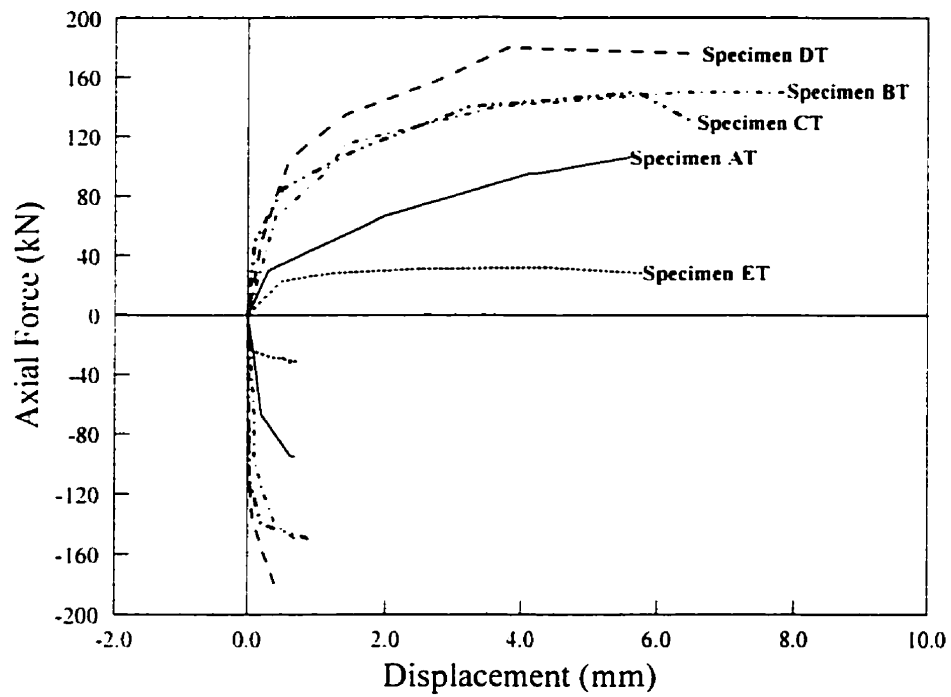


Figure 6.4 Envelopes of hysteresis responses of Specimens AT through ET

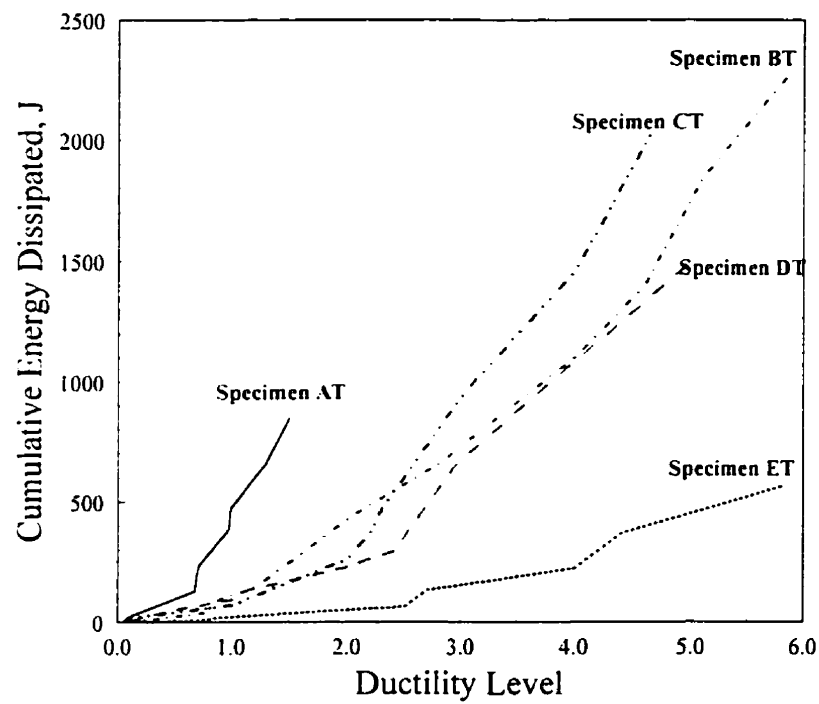


Figure 6.5 Cumulative hysteresis energy dissipation of Specimens AT through ET

6.2.2.3 Response Comparison of Connections AS and ApS

The Specimen ApS was identical to the Specimen AS, except that the reinforcing steel anchor was welded to 200 x 50 x 10 mm steel plate instead of a steel angle. This was designed to evaluate the behaviour and response parameters of the connection with a steel plate, and also to investigate the mechanism of the possible fracture failure of the weld between the steel plate and the reinforcing steel anchors. The Specimen ApS was tested under reversed cyclic shear loads.

The nature of the hysteresis response loops for both Specimens AS and ApS were similar, see Figures 3.5 and 3.32. The Specimen ApS showed stiffer response up to the maximum load carrying capacity. However, upon commencement of the fracture of the weld between the steel plate and anchor, significant degradation in both stiffness and strength were noticed. The stiffness and strength for the Specimen ApS kept on decreasing through the repeating load cycles, which was due to the gradually increasing fracture of the weld.

Table 6.4 compares the cracking , yielding and ultimate strengths as well as the failure modes of the various specimens. The Specimen ApS shows a smaller cracking load, which was due to the presence of the steel plate instead of the steel angle as in the Specimen AS. The area of the steel angle which was bonded to the concrete was larger than that for the steel plate, which caused more load be resisted by the Specimen AS compared with the Specimen ApS. Thus the Specimen AS was stiffer in the early stage of the loading. In term of the yielding load capacities, the Specimen ApS showed a lower yield load. This was due to rotation and bending of the anchor at the location of 45° bent caused by the shearing load acting at an eccentricity between the weld and the loading line. This phenomenon resulted in higher strain being induced in the anchor and reduced the load required to yield the anchor. Having the anchor welded along the flange of the steel angle in the Specimen AS prevented this rotation and bending of the anchor

The envelopes of hysteresis responses for Specimens AS and ApS are compared in Figure 6.6. The envelopes in Figure 6.6 confirm the severity of the strength degradation for Specimen ApS. This was a consequence of the weld tearing in Specimen ApS. Beyond the 80 kN shear force, the Specimen AS showed a more flexible response

envelope (see Figure 6.6) which was due to complete debonding of steel angle flanges from the surrounding concrete. The ultimate strength of the Specimen ApS (153 kN) was almost the same as that for the Specimen AS (150 kN), the difference being less than 2%.

The mode of failure for the Specimen ApS was due to fracture of the weld between the steel plate and the reinforcing anchor [Figure 3.35(b)].

Figure 6.7 compares the cumulative energy dissipated versus the measured displacement. Up to displacement of 3.0 mm, the cumulative energy dissipated was similar for both specimens, as shown in Figure 6.7. The total cumulative dissipated energy was greater for Specimen AS compared with that for the Specimen ApS, however, the same ultimate displacements were measured for the specimens. The smaller dissipated energy in the Specimen ApS can be attributed to the stiffer response before the post-peak loading and the hysteresis behaviour which was affected by progress of the weld fracture during repeated loading cycles. The displacement ductility ratio for the Specimen ApS ($\mu = 5$) was greater, which was related to the smaller yield displacement of Specimen ApS ($\Delta_y = 1.4$ mm) compared with that for Specimen AS ($\Delta_y = 2.3$ mm).

Connection Types	Cracking load (kN)	Yielding load (kN)	Ultimate strength (kN)	Failure modes
AS	77	150	150	Failure along the tensile leg and the along diagonal cracks
ApS	65	145	153	Fracture failure of the weld between steel plate and bar anchor

Table 6.4 Comparison of response parameters of Specimens AS and ApS

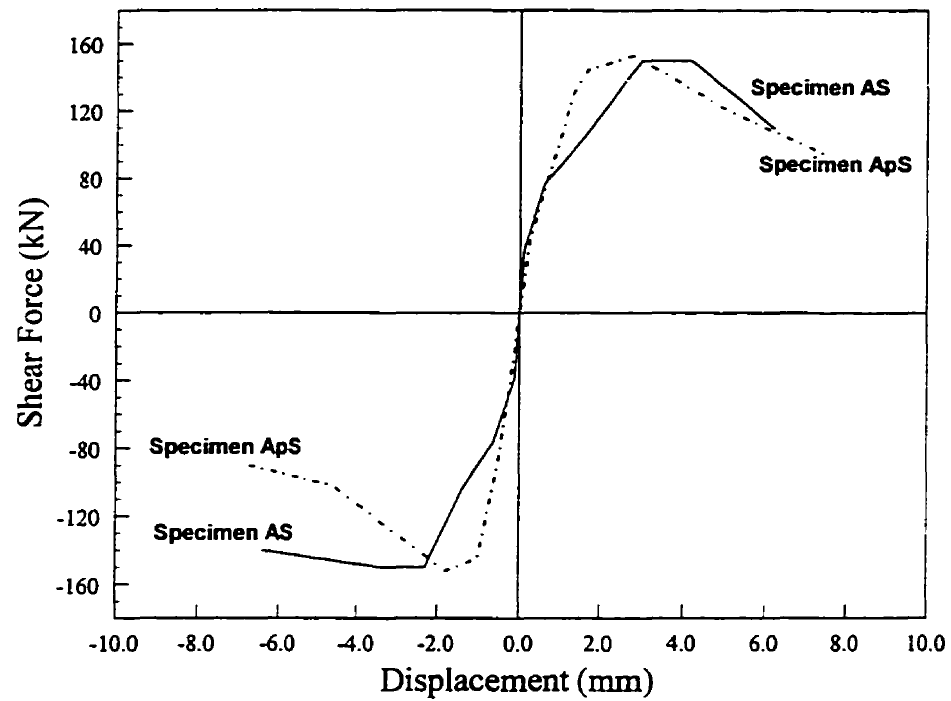


Figure 6.6 Envelopes of hysteresis responses of Specimens AS and ApS

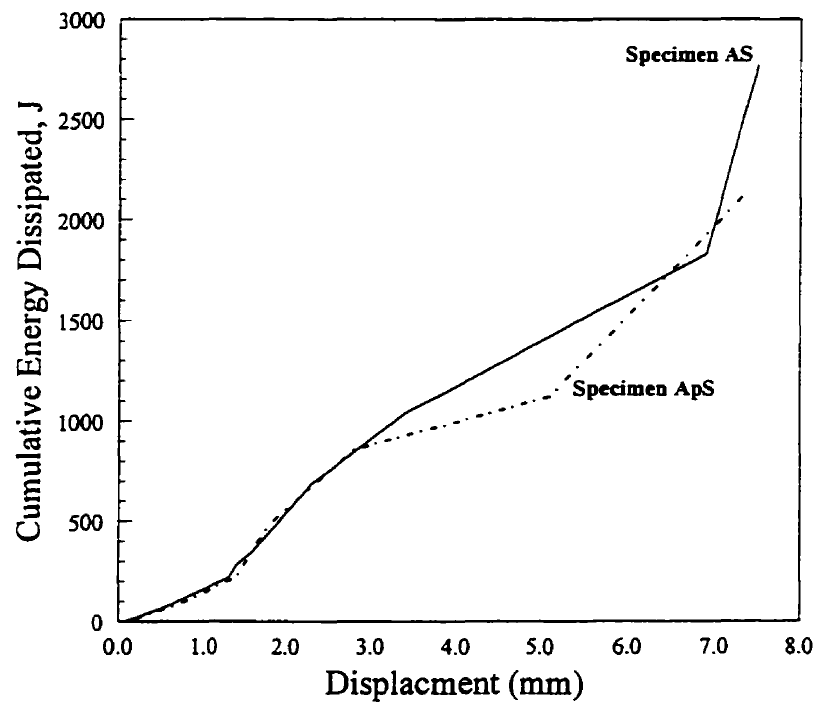


Figure 6.7 Cumulative hysteresis energy dissipation of Specimens AS and ApS

6.3 Comparison Between Specimens in Each Series

In this section, the response of identical specimens subjected to different loading conditions (monotonic increasing or reversed cyclic loads), and the different combination of cyclic shear and axial loads (T/V ratios) are compared to determine the effects of these variables on the overall response of each connection type. There is more emphasis on discussion of the different aspects under monotonically increasing (monotonic loads) and reversed cyclic shear loading (cyclic loads) and the effects of cyclic axial load on the cyclic shear responses, and vice versa.

6.3.1 Series A Specimens

6.3.1.1 Comparison of Monotonic and Cyclic Shear Responses

Figure 6.8 compares the shear force - lateral displacement responses of the connection type A subjected to monotonically increasing and reversed cyclic shear forces (Specimens AS and AMS). Figure 6.8 shows that the responses of the connection during the earlier stages of loading (up to 70 kN) which were elastic and basically identical for both Specimens AS and AMS, suggesting that in the elastic region of loading, the cyclic load did not decrease the connection stiffness and strength. Beyond the elastic stage, the degradation of stiffness and increase of displacement were larger for the cyclic loads than for the monotonic loading. This was a consequence of the crack progression and bond deterioration under cyclic loading. A sudden drop in the load after the maximum strength attained for the monotonic response was due to rapid crushing and spalling of the concrete in front of the steel angle, which was not the case for the cyclic loading due to the gradual spalling and crushing during the various cycles.

The cracking, yielding and the ultimate strengths along with the various failure modes for monotonic and cyclic shear loading are listed in Table 6.5. The cracking load for the cyclic loading (77 kN) was less than that for the monotonic loading (85 kN), see Table 6.5, which was a consequence of initiation and propagation of micro-cracks during the reversed loading cycles. For monotonic loading, the yielding occurred after the maximum strength was attained and partial failure occurred due to the extensive spalling-cracking of the concrete. The compression leg did not offer much resistance and the

Loading type	Monotonically increasing shear loads				Reversed cyclic shear loads				β
Connection type	C.L. (kN)	Y.L. (kN)	U.L. (kN)	Type of failure	C.L. (kN)	Y.L. (kN)	U.L. (kN)	Type of failure	
A	85	110	187	Crushing-spalling along the length of the anchors	77	150	150	Failure along diagonal cracks (conical failure surface)	0.80
B	95	165	242	Spalling-splitting along the tension leg plus crushing-spalling in front of the hook	82	150	200	Spalling-splitting along both the anchor legs plus crushing-spalling in front of the hook	0.83
C	70	125	152	Splitting failure along anchor in the tension zone extended toward the centre	65	105	125	Complete failure in concrete surrounding anchor legs close to the steel angle	0.82
D	70	205	226	Failure along the diagonal cracks formed at end and at mid-length of tensile anchor	65	150	190	Failure along the diagonal crack formed at end of the tensile leg	0.84
E	29	45	53	Shear fracture of both studs	15	25	46	Shear fracture of both studs	0.87

Note: C.L. , Y.L. and U.L. represent Cracking Load, Yielding Load and Ultimate Load, respectively.

Table 6.5 Comparison of monotonic and cyclic shear response parameters for Connections A through E

shearing force was sustained basically by the tension leg, thereby increasing the steel strains considerably with the bar yielding at a load of 110 kN. However, for cyclic loading since there was gradual deterioration of the concrete in the compression zone, the reinforcing bar yielded at load of 150 kN. The corresponding yield displacement was 2.3 mm, which is smaller than that for the monotonic loading ($\Delta_y=3.0$ mm).

The crack pattern was different because under reversing cyclic loads the stresses at a particular location in the concrete changed alternately from compression to tension, and vice versa. This affected on the pattern of cracks and their propagation during cyclic testing. Also, the number of the cracks was higher for the cyclic loads than that for the monotonic loading. Type A connection attained ultimate strengths of 187 kN and 150 kN (see Table 6.5) when subjected to monotonically increasing and reversed cyclic loading, respectively. This showed that the strength of the Type A connection under the cyclic loads reduced to about 80% of that for the monotonic loading. The ratios of β = cyclic ultimate strength / monotonic ultimate strength for all connection types are presented in Table 6.5. The ultimate displacements were 5.2 mm and 7.4 mm for the monotonic and the cyclic loading cases, respectively, indicating that the ultimate displacement under cyclic loads increased to about 1.4 times that for the monotonic loads. Larger displacement ductility ratio was achieved for cyclic loads ($\mu = 3$) in comparison with that for monotonic loading ($\mu = 1.7$).

Although both Specimens AS and AMS failed due to the concrete deterioration, the final failure modes were different. The final mode of failure under reversed cyclic loading was due to failure along the diagonal cracks (conical failure surface), while for the monotonic loading it was due to crushing- spalling along the both leg of the reinforcing anchor, which was brittle, see Figures 3.8(b) and 3.4(b). In addition, the length of the anchor bar that yielded at ultimate load under cyclic loading (4/5 of the bar length) was about 2.5 times that for monotonic loading (1/3 of the length). The crushed concrete in front of bar was trapped between the bar and the surrounding concrete. This retention of the crushed concrete at the interface, and its continuing contribution to the strength of the connection under monotonic loads partially explains the behaviour of the connection which failed suddenly (significant drop in load) at a load close to the

maximum strength accompanied with a small deflection and brittle failure.

6.3.1.2 Cyclic Response under Combined Shear and Axial Forces

Identical type A connections were subjected to different ratios cyclic axial load to cyclic shear load (T/V ratios) to study the responses under combined cyclic loadings and to investigate the effect of cyclic axial force on the shear responses, and vice versa.

Hysteresis responses , strengths and displacements

When 30% of the cyclic shear forces were applied in the orthogonal direction as an axial force simultaneously with the shear (T/V ratio = 0.3, Specimen AST3), the nature of the hysteresis shear response (Figure 3.25) was similar to that for the pure cyclic shear loading (Specimen AS, Figure 3.5). This was due to application of small values of cyclic axial loads; the ultimate shear strength of the connection reduced to about 93% of that attained under pure cyclic shear loading (Table 3.6). The shear stiffness of the connection reduced as more with 30% of the shear was applied as cyclic axial force when compared to that for pure cyclic shear response (Specimen AS). The horizontal displacement in the positive direction (combined shear and tensile axial forces) had increased slightly compared with that for Specimen AS as a consequence of cyclic axial loads application.

When 70% of the shear force was applied as axial force in the orthogonal direction (T/V ratio= 0.7, Specimen AST1), the shear hysteresis response was different from that for Specimen AS due to effects of axial forces (Figures 3.13 and 3.5). The nature of the axial hysteresis response curve was similar to that for Specimen AT (pure cyclic axial loading), see Figures 3.14 and 3.9. The displacement had increased compared to the response of Specimen AS at a similar load value. This was due to the application of moderate cyclic axial load which induced more distress in the concrete and the reinforcing anchor and caused larger displacements. The ultimate strengths for cyclic load combination ratio of T/V=0.7 was reduced to about 60% and 57% of those for pure cyclic shear (AS) and pure cyclic axial loads (AT), respectively, as shown in Table 3.6.

The Specimen AST2 subjected to combined cyclic loading with a T/V ratio =1.0 revealed different hysteresis response compared to the response of the Specimen AS

(Figures 3.25 and 3.5). The large loops in the positive loading direction (combined shear and tensile axial loads) and very narrow loops in the negative direction due to compressive axial load was the major difference compared with the Specimen AS. The same load -deflection pattern was noted for the shear response compared with the response of the Specimen AST1 with minor difference for the loops in negative direction (Figures 3.19 and 3.13). The loops were narrower in the shear response of Specimen AST2 ($T/V=1.0$) due to the higher cyclic compressive axial loads. The Specimen AST2 exhibited similar shear and axial hysteresis response, while the shear and the axial responses were not similar to each other for the specimens with lower values of the T/V ratio (Specimens AST1 and AST3). The nature of the axial hysteresis for Specimen AST2 ($T/V=1.0$) was different from that for the Specimen AT. This implies that the effects of cyclic shear forces on the axial loading response caused different response characteristics. The ultimate strengths under combined cyclic loading with ratio of $T/V=1.0$ reduced to about 50% and 72% of those for pure cyclic shear and axial loading, respectively. It indicated that for the Type A connection the effect of cyclic axial forces on the shear capacity reduction was more severe than the influence of the cyclic shear forces on the degradation of the axial strength. The displacements in the positive direction for the Specimen AST2 ($T/V=1.0$) had larger values compared with those for the Specimens AS, AT, AST3 ($T/V=0.3$) and AST3 ($T/V=0.7$) at the same load values.

For all combined cyclic loading cases (Specimens AST1, AST2, AST3), the reduction of the shear strengths as compared to the pure cyclic shear (see Table 3.6) due to the axial cyclic loads, resulted in the additional stressing of the bond region, thus weakening the bond strength for succeeding applications of the shear force, resulting in a decrease of strength.

Crack pattern, failure mode and anchor strain

The crack patterns for the Specimens AST1, AST2 and AST3 were similar, however, they were different from the crack patterns for the pure cyclic loading (Specimens AS and AT). This is due to the effect of the axial load which changed the stress distribution in the concrete, resulting in different crack patterns.

The final modes of failure for all of the specimens in Series A, subjected to

combined cyclic loading were similar and was mostly due to the spalling-splitting along the tensile anchor leg accompanied with the disintegration of the concrete surrounding the steel angle, as shown in Figures 3.18 (b), 3.24 (b) and 3.30 (b). The failure mode for the Specimen AT was close to those for the combination loading cases (Figure 3.12(b)). However, the mode of failure for Specimen AS was different from those for the other specimens in Series A (Figure 3.8(b)). This was attributed to the excessive bond stresses at the concrete-steel interface in the tensile zone due to the cyclic axial load being applied simultaneously with the shear forces. The reversed cyclic axial load weakened that area causing the mechanism of failure to form, and reduced the overall strength of the connection. Also, the measured axial strains along the reinforcing anchors at ultimate load for specimens in the Series A show that by increasing the ratio of T/V , the yield length of the reinforcing anchors increased, (see Figures 3.17, 3.23 and 3.29).

Energy dissipation and ductility level

Figure 6.9 shows the cumulative energy dissipation versus the ductility level for all of the specimens in Series A. As can be seen, the Specimen AS subjected to pure cyclic shear load attained the greatest cumulative dissipated energy and ductility level. The Specimen AST3 with a low ratio of $T/V=0.3$ dissipated more energy compared with Specimens AST1 ($T/V=0.7$) and AST2 ($T/V=1.0$), see Figure 6.9. This was attributed to the hysteresis responses, especially the hysteresis shear response. Since the hysteresis shear response of Specimen AST3 consisted of larger loops due to the higher shearing forces compared to the Specimens AST1 and AST2, the cumulative dissipated energy was greater. The Specimens AST1 had the capability to dissipate energy close to that for the Specimen AST2 (Figure 6.9). Thus, when the cyclic axial load was applied simultaneously with the cyclic shear in Type A connection, the capability of the connection to dissipate energy is significantly reduced.

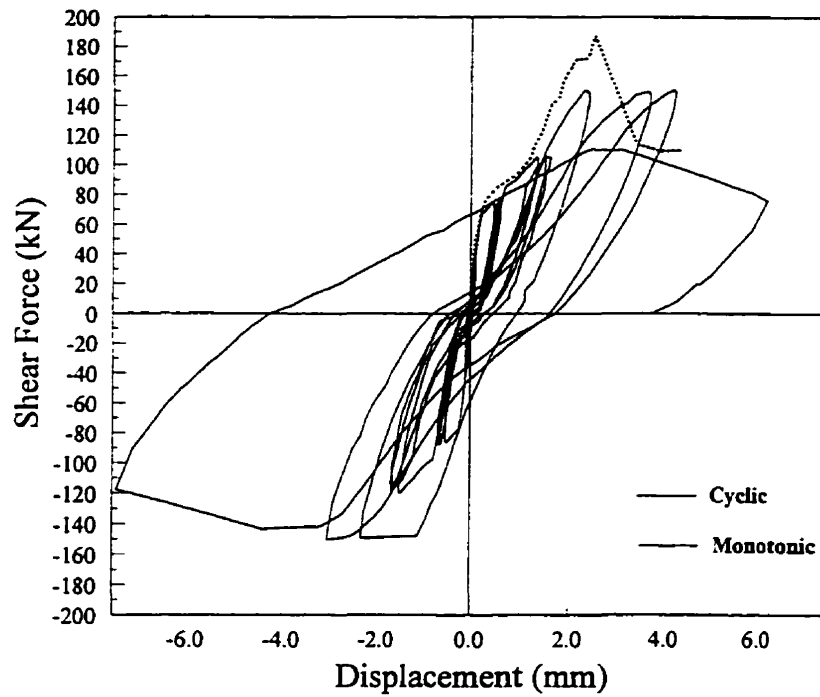


Figure 6.8 Comparison of monotonic and cyclic responses for specimens in Series A (Specimens AS and AMS)

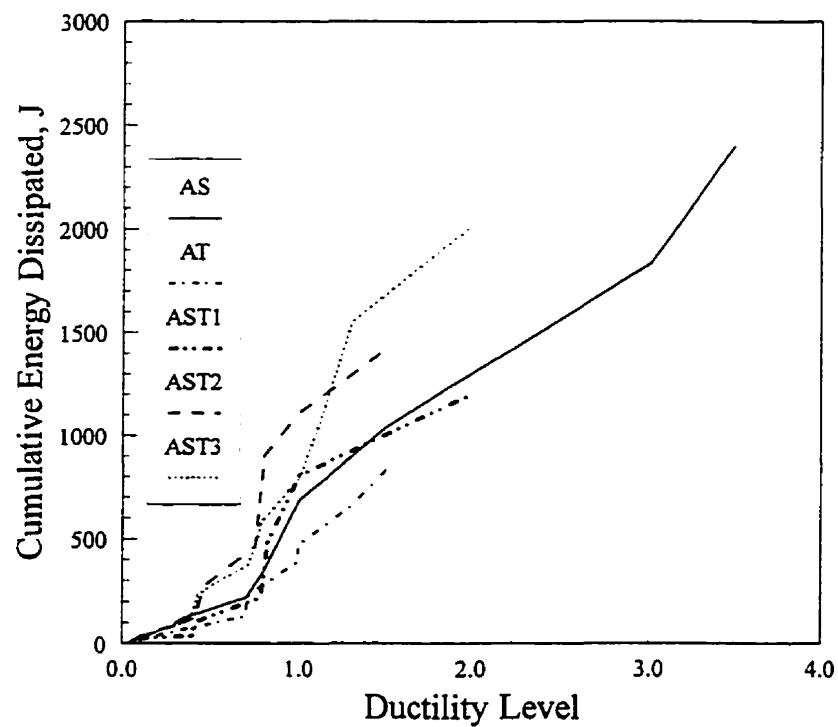


Figure 6.9 Comparison of cumulative dissipated energy for specimens in Series A

6.3.2 Series B Specimens

6.3.2.1 Comparison of Monotonic and Cyclic Shear Responses

Figure 6.10 compares the responses of the specimens in Type B connection tested under monotonically increasing and reversed cyclic shear forces (Specimens BS and BMS). Up to a load of 80 kN, the slope of the load-deflection curve remained basically constant and the same for both the monotonic and the cyclic loads (Figure 6.10). This showed that the loading cycles in elastic stage did not impair stiffness and strength of the connection. Beyond the cracking load, the stiffness degradation was more significant for the cyclic loading compared with that for the monotonic response. This was attributed to the crack propagation in the concrete and deterioration of the bond strength at the steel anchor-concrete interface due to the cyclic load.

The cracking load for monotonic loading (95 kN) was larger than that for cyclic loading (82 kN, see Table 6.5), which was due to initiation of the micro-cracks and their progression under reversed cyclic loads. The yielding strength for cyclic loading (150 kN) was less than that for monotonic loading (165 kN). The higher yield strength for the monotonic loading was in fact related to relatively higher "tension stiffening" caused by concrete surrounding the steel anchor. However, due to the reversed cyclic loading, the bond strength deteriorated significantly resulting in great reduction of the "tension-stiffening" effect due to a decrease in the yield strength compared with the monotonic loading case.

The crack patterns under monotonic and cyclic loading were different, since the reversed loading cycles changed the distribution of the stresses from compression to tension, and vice-versa. This influenced the crack pattern and density, which were more extensive under cyclic loading.

The Type B connection achieved ultimate strengths of 242 kN and 200 kN for the monotonic and cyclic loading cases, respectively, as shown in Table 6.5. This showed that under reversed cyclic loading for type B connection, the ultimate strength decreased to 83% of that for the monotonic loading, and given (as factor β) in Table 6.5. This was a consequence of bond strength deterioration and exhaustion of the concrete strength in front of 90° hooks under reversed cyclic stresses induced in the connection by the cyclic

loading. In addition, under monotonic loads, the crushed concrete in front of the straight part of the anchor (concrete-anchor interface) and 90° hook was trapped between the anchor (both straight part and the hook) and the surrounding concrete, resulting in retention of the crushed concrete at those locations, and its continued contribution to the resistance of the applied load, which explains the partially higher strength for the monotonic loading condition. However, during the cyclic loads the concrete did not remain trapped due to reversal of the resulting stresses from compressive to tensile and vice versa, which resulted in its insignificant contribution to the load carrying mechanism. The ultimate displacement under monotonic loading (6.8 mm) was about 1.5 times of that for the cyclic loading (4.5 mm), giving displacement ductility ratio of 3.8 and 2.6 for monotonic and cyclic loading, respectively.

Failure of the Specimen BMS subjected to monotonic loads occurred in the tension zone due to spalling-splitting along the leg and spalling-crushing in front of the 90° hook, while for the cyclic loading, failure occurred along both legs and both hooks. This was again because of the reversing loading cycles which weakened the concrete surrounding both anchor legs, resulting in development of failure mechanism in both areas (right and left sides of the connection). Furthermore, the length of the steel anchor yielded at ultimate load under cyclic loading (1/2 of the bar length) was about 1.4 times that for the monotonic loading (2/3 of the bar length).

6.3.2.2 Cyclic Response under Combined Shear and Axial Forces

The response parameters of identical Type B connection specimens, subjected to different T/V ratios are discussed and compared. This provides the necessary data to establish the design criteria and equations for determining the strength of the connection subjected to the various combinations of loads resulting from the seismic actions.

Hysteresis responses, strengths and displacements

The nature of shear response under the application of 30% of cyclic shear force applied in the normal direction simultaneously with the shear load (Specimen BST3) was similar to the response for pure cyclic shear loads (see Figures 3.56 and 3.36). This was related to application of a small axial force, which did not significantly change the

stresses induced in the connection by the shearing forces, and therefore, they did not have any significant effect on the nature of the shear response. However, the ultimate shear strength of connection decreased to about 75% of the pure cyclic shear strength, as shown in Table 3.11. The slope of the shear load - horizontal displacement curve (connection stiffness) of Specimen AST3 reduced more throughout the cycles when compared with that for the Specimen BS, which was the consequence of the effect of the axial load on the connection resulting in an increase in its flexibility. The horizontal displacements in the positive direction (combined cyclic shear and tensile axial) had increased compared with those for pure cyclic shear (see Figures 3.56 and 3.36). This was caused by the axial load which caused more deterioration in the concrete at the particular load stage and in turn resulted in an increase of the displacements.

With an increase in the percentage of cyclic axial load to 70% of the shear load ($T/V=0.7$, Specimen BST1), the shape of the shear response differed from that for the Specimen BS (see Figures 3.36 and 3.44). This revealed that for the T/V ratios larger than 0.7, the shear response characteristics were changed. This was related to the different stresses distributions and transformation in the connection due to application of the larger axial loads. There was more pinching in the load-deflection response of the Specimen BS compared with those for the Specimens BST1 and BST2, since the axial load caused larger residual displacement upon unloading to zero, thereby decreasing the pinching. The nature of the axial hysteresis response was different from that for the Specimen BT (pure cyclic axial loading) due to the effects of the application of the cyclic shear forces. The ultimate horizontal displacement for BST1 was larger than that for the Specimen BS at a similar load value. A comparison of horizontal displacements between the Specimen BST1 ($T/V=0.7$) and the Specimen BST3 ($T/V=0.3$) showed larger horizontal displacement for the Specimen BST3. However, the ultimate axial displacement was larger for Specimen BST1 compared with that for the Specimen BST3 (Figures 3.44 and 3.56).

The ultimate strengths under combined loading with T/V ratio $=0.7$ show that the connection strengths decreased to about 70% and 47% of those for pure cyclic shear and axial loads, respectively, as shown in Table 3.9.

The Specimen BST2 subjected to combined cyclic shear and axial forces, with

T/V ratio =1.0, indicates that the hysteresis response was different from that for the Specimen BS (see Figures 3.36 and 3.50); very narrow loops in the negative direction (combined shear and axial compression loads) for shear response in comparison with the Specimen BS was due to the effect of the high compressive axial loads which reduced the horizontal displacements in the negative shearing direction. The nature of the hysteresis responses for the Specimen BST2 (T/V=1.0) and BST1 (T/V=0.7) were similar (see Figures 3.50 and 3.44). The shape of the shear and the axial hysteresis responses for the Specimen BST2 were almost similar, the main difference being that for the axial hysteresis response, the loops in the negative loading direction were narrower and remained positive compared to those for the shear hysteresis response (see Figures 3.50 and 3.51). This was different from the responses of the Specimens BST1 (T/V=0.7) and BST3 (T/V=0.3), which did not show significant similarity between the shear and the axial responses. The pattern of the axial hysteresis response of Specimen BST2 (T/V=1.0) was not similar to the response of the Specimen BT (pure cyclic axial load), as shown in Figures 3.51 and 3.40. This was attributed to the effects of the application of the cyclic shear forces on the distribution of the stresses in the connection, which influence the axial hysteresis parameters significantly.

The ultimate horizontal displacement was smaller than those for Specimens BST1 (T/V=0.7) and BST3 (T/V=0.3), however, the ultimate axial displacement was larger than those with smaller value of T/V ratio (Specimens BST1 and BST3). Therefore, a comparison the of ultimate displacements for the different T/V ratios indicates that with an increase in the percentage of the cyclic axial forces (increase in the T/V ratio), the horizontal displacements decreased and the axial displacement increased. This was related to the connection details, since the hooks were more effective with the axial loads (both hooks are subjected to tensile stresses), thus an increase in the cyclic axial loads had more influence on the axial displacements.

The results for the combined loading case with T/V ratio =1.0 ratio shows that the strength of the connection reduced to about 50% and 67% of those for pure cyclic shear (BS) and pure cyclic axial (BT) strengths, respectively. It showed that for the type B connection, the influence of axial forces on the degradation of the ultimate shear strength

was more severe than the effect of the shear force on the reduction of the ultimate axial strength. The reduction of strengths in the different load combinations, compared to pure cyclic loading, were due to the additional stresses induced in the bond region and in front of the 90° hooks by the cyclic axial load, thus weakening the strength of those locations for succeeding application of the shear forces (following cycles). The same phenomenon explains the degradation of axial strength due to the shear forces.

Crack patterns, failure mode and anchor strain

The specimens subjected to combined loadings (BST1, BST2 and BST3) exhibited similar crack patterns. However, the crack pattern for the specimens under pure cyclic loading were different from those subjected to combined loads (different T/V ratios). The main difference was that for the combined loading, the density of the cracks in the tension zone was more than that in the compression zone, while the crack pattern for the pure cyclic loading were symmetrical. This is a consequence of the combination of cyclic shear and axial loading, which resulted in one side of the connection always being subjected to tensile stresses and the other side being subjected to the resultant stresses induced by combination of cyclic shear and axial forces. However, for the pure cyclic shear loading, the stresses changed alternately from tensile to compressive stresses and vice-versa, while for the pure cyclic axial load, the entire connection was alternately subjected to tensile and compressive stresses.

The modes of failure for the combined loading cases were similar, and the failure was due to splitting- spalling along the tensile leg, broken concrete surrounding the steel angle and crushing of the concrete in front of the 90° hook in the tension zone, as shown in Figures 3.49(b), 3.56(b) and 3.61(b). The main difference between the final failure mode of the Specimens BT (pure cyclic axial) and DS (pure cyclic shear) was the symmetric failure which was not symmetric for the combined loading cases (see Figures 3.39(b), 3.43(b), 3.49(b), 3.56(b) and 3.61(b). For the combined loads, the reinforcing anchor in the tension zone was subjected to higher tensile stresses which weakened the bond strength at the concrete-steel interface and the concrete in the front of the 90° hook. Under reversed cyclic loads, this phenomenon induced reversed stresses in those area and developed the failure mechanism. The strain profiles along the anchor length for the

combined loading cases at the ultimate load revealed that the higher T/V ratio resulted in a larger length of the reinforcing bar which yielded (see Figures 3.48, 3.55 and 3.60).

Energy dissipation and ductility level

Figure 6.11 compares the cumulative energy dissipation versus the ductility level for the specimens in the Series B. The Specimen BS showed the smallest cumulative dissipated energy and ductility level. The specimen BST3, with T/V ratio =0.3, dissipated more energy and had larger displacement ductility compared with those for the Specimen BST2 (T/V=1.0) and BST1(T/V=0.7). This was related to the hysteresis responses, specially the shear responses, and since the hysteresis shear response of the Specimen BST3 consisted of larger loops, resulting in greater cumulative energy dissipation. The Specimens BST2 (T/V=1.0) and BST1 (T/V=0.7) indicated almost the same capability to dissipate energy and attain the ductility level, as shown in Figure 6.11. Thus application of higher T/V ratios results in lower energy dissipation in the type B connection.

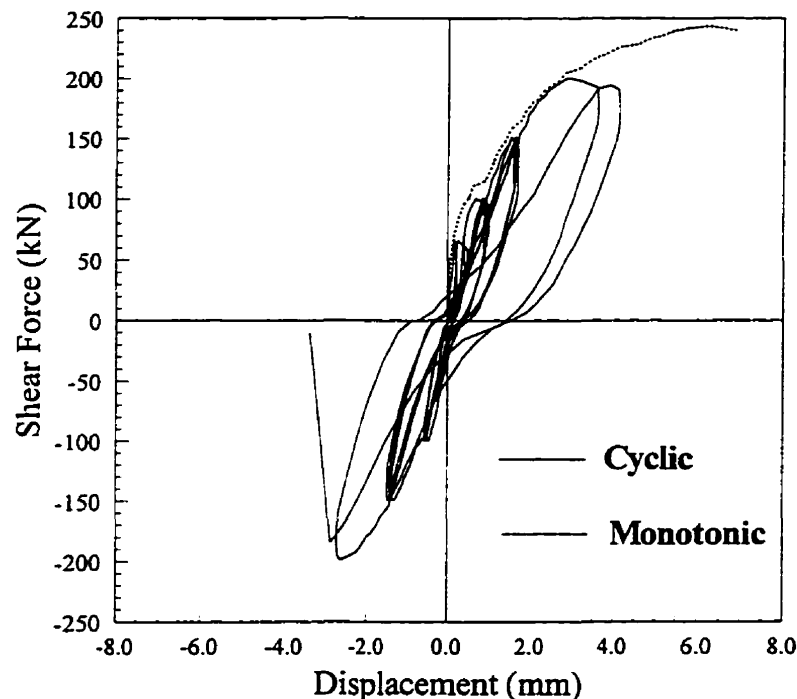


Figure 6.10 Comparison of monotonic and cyclic responses for specimens in Series B (Specimens BS and BMS)

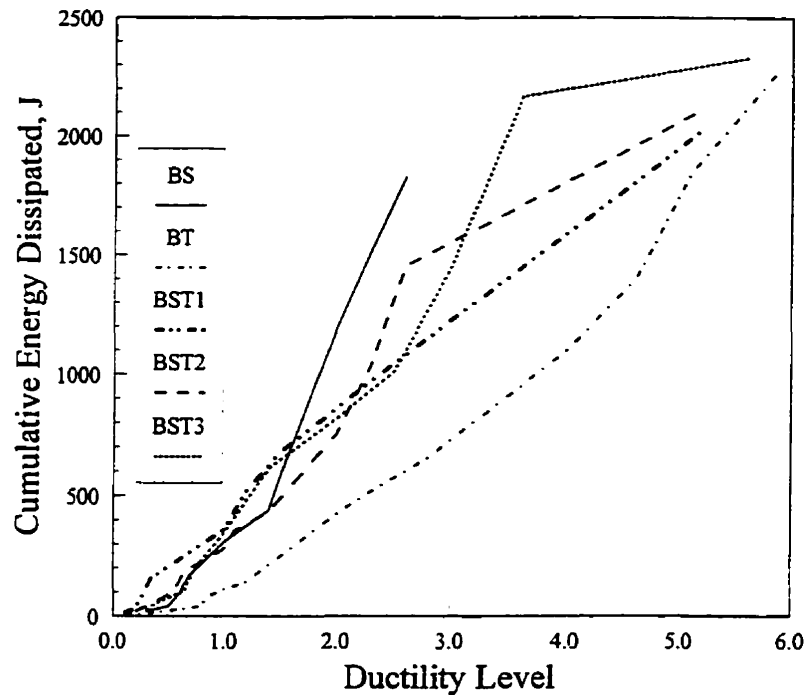


Figure 6.11 Comparison of cumulative dissipated energy for specimens in Series B

6.3.3 Series C Specimens

6.3.3.1 Comparison of Monotonic and Cyclic Shear Responses

Figure 6.12 compares the responses of Type C connections (Specimens CS and CMS) subjected to monotonically increasing and reversed cyclic shear forces. The load-deflection response remain linear up to Cycle 4A ($P=65$ kN), and it was coincident with the monotonic response, as shown in Figure 6.12. It confirms that in the elastic response region, the cyclic loading did not decrease the stiffness and strength of the connection. Beyond the cracking load, the reduction of the connection stiffness was more significant in the cyclic response compared with that for the monotonic loading. This was the results of propagation of more cracks in the concrete and increasing bond strength deterioration at the concrete-steel anchor interface due to the cyclic loading.

The cracking load for the cyclic loading (65 kN) was smaller than that for the monotonic loading (70 kN), see Table 6.5. This was a consequence of the initiation and propagation of micro-cracks during the elastic stage of reversed cyclic cycles resulting in

a lower cracking strength. The yielding load under monotonic loading (125 kN) was larger than that for the cyclic loading (105 kN), as shown in Table 6.5, which was due to the extensive cracking of the concrete surrounding the anchor which was subjected to cyclic loads prior to yielding. The cracked concrete reduces the effect of the concrete contribution to the "tension stiffening" of the anchor and decreases the yield strain compared with that for the uncracked concrete. In addition, for the monotonic response, the crushed concrete in front of steel anchor (due to dowel action) was trapped between the bar and the uncracked concrete and its retention at the interface continued to contribute to the resistance to the applied load, while for the cyclic load, the crushed concrete at the interface was already cracked in the previous cycles and in addition the crushed concrete was not significantly trapped by the surrounding cracked concrete. Thus the reversed cyclic loading decreased the load required to yield the anchor compared with that for the monotonic loading. The corresponding yield displacement was $\Delta_y=1.3$ mm, which was smaller than that for monotonic loading ($\Delta_y=2.4$ mm). The cracking pattern was distinguished as being symmetrical for cyclic loading and unsymmetrical for the monotonic loading due to the reversal of the stresses induced in the connection by the loading cycles. The density of cracks was more significant for the cyclic loading , even for the left part of the connection, due to reversing stresses and cracking which influenced the succeeding crack patterns. In addition, the concrete spalling-crushing along the anchor close to the steel angle occurred during the cyclic loading, which was not the case for the monotonic loading.

The ultimate strength under reversed cyclic loads (125 kN) for type C connection reduced to 82% of that for the monotonic loading (152 kN), as shown in Table 6.5. The higher values for the monotonic loading are explained based on the same phenomenon described for the higher value of the yielding load compared with the cyclic response. It must be also noted that the development of yielding along both reinforcing bar anchors in turn influenced the degradation of the ultimate strength significantly compared with that for the monotonically increasing loads. The ultimate monotonic displacement (9.0 mm) was about 1.4 times of that for the cyclic loads (6.6 mm), giving displacement ductility ratios of 3.7 and 5.1 for monotonic and cyclic loads,respectively, with larger

ductility ratios for the cyclic loading.

The final mode of failure under cyclic loading was completely different from that for the monotonic loading [see Table 6.5 and Figures 4.8 (b) and 4.4 (b)]. The basic difference was due to the reversed cyclic loading (reversed cyclic dowel action) which induced reversal of stresses in the region close to the steel angle resulting in the loss of the strength of the concrete surrounding the anchor and finally leading to the failure mechanism.

Moreover, the length of steel anchor which yielded at the ultimate load under cyclic loading ($3/4$ of the length) was larger than that for the monotonic loading ($2/3$ of the bar length).

6.3.3.2 Cyclic Response under Combined Shear and Axial Forces

The hysteresis responses, ultimate strengths, displacement ductilities and energy dissipation capabilities of identical specimens (Type C connection or Series C) subjected to the different combination of cyclic shear and axial loads (different T/V ratios) are presented in this section. The characteristic and response parameters are compared to develop the interaction equation and guidelines for design of connections subjected to earthquake forces.

Hysteresis responses , strengths and displacements

For the Specimen CST3 (T/V ratio =0.3) with a small value of axial load applied simultaneously in the normal direction, the nature of the hysteric shear response was similar to that for the pure shear loading [Figures 4.5(b)) and 4.22(a)]. The main difference was the larger loops for the Specimen CST3 in the last stages of loading due to the application of cyclic axial loads. This indicates that the axial load, even at small value, was significantly effective beyond the yielding and the extensive cracking stages. The ultimate shear strength of the connection did not decrease significantly compared with that for the pure cyclic shear loading (only 96%, as shown in Table 4.6). This shows that the simultaneous application of the cyclic shear loading in the normal direction up to 30% of the cyclic shear forces, the shear strength of the connection was not noticeably affected. The stiffness of the connection reduced due to the application of

even small axial loads, which was a consequence of the additional cracking corresponding to the cyclic axial loads. The ultimate horizontal displacement in the positive direction (combined cyclic shear and tensile axial loads) for the Specimen CST3 had increased considerably compared with that for the pure cyclic shear (Specimen CS, see Figures 4.5 and 4.25). This was again the result of the axial stresses induced in the connection by the cyclic axial loads causing more cracking, which influenced the material property parameters, i.e. more cracks and yielding decreased the elastic modulus and the strength of the connection, which in turn reduced the stiffness of the connection.

Increasing the ratio of the cyclic axial load to about 70% of the applied cyclic shear load (T/V ratio =0.7, Specimen CST1), resulted in a different shear response compared to that for the Specimen CS (Figures 4.5 and 4.13). The pinching was severe for the Specimen CST1 compared with the Specimen CS due to effects of the moderate cyclic axial forces (70% of the shear load). The larger loops for the positive loading direction and significant strength degradation were the important difference between the Specimen CST1 and the Specimen CS. In fact, these characteristics were due to the effect of the cyclic axial forces, which increased the rate of concrete deterioration, increasing the strength degradation rate and forming larger hysteresis loops. The pattern of hysteresis axial response for the Specimen CST1 differed from the Specimen CT (pure cyclic axial loading), see Figures 4.14 and 4.9. This can be attributed to the effects of cyclic shear forces on the connection axial response. The ultimate horizontal displacement of the Specimen CST1 ($T/V=0.7$) was larger than those for the Specimens CS and CST3 ($T/V=0.3$), see Figures 4.13 and 4.5 and 4.25; and also the ultimate axial displacement of the Specimen CST1 was larger than that for the Specimen CST3 (See Figure 4.14 and 4.26). The ultimate strengths under combined cyclic loads with T/V ratio =0.7 showed considerable reduction in the connection strengths. The findings show that the strength of the connection reduced to about 80% and 45 % of pure cyclic shear (CS) and pure cyclic axial (CT) strengths, respectively (see Table 4.6).

The Specimen CST2 tested for a T/V ratio =1.0, showed a different hysteresis shear response in comparison with the Specimens CS and CST3 ($T/V=0.3$), while it was similar to that for the Specimen CST1 ($T/V=0.7$). The only difference between the

response of the Specimens CST2 and CST1 was the strength degradation through the cycles which was more for the Specimen CST2 compared with that for the Specimen CST1. Comparing the degradation of the strength for the type C connection, it was found that with an increase of the T/V ratio, the rate of strength degradation due to cyclic loads increased. There was not significant similarity between the shear and the axial hysteresis responses for all of the specimens subjected to combined cyclic loads. The nature of the axial hysteresis response of the Specimens CST1, CST2 and CST3 were different from the response of the Specimen CT (pure cyclic loading). This shows that the high value of cyclic shear changed the nature of the responses compared with the Specimen CT. The ultimate horizontal displacement of the Specimen CST2 ($T/V=1.0$) was slightly larger than that for the Specimen CST1 ($T/V=0.7$). Therefore, an increase in the T/V ratio, increases the ultimate displacements. Based on the experimental results for the combined cyclic axial and shear forces with T/V ratio =1.0, the strength of the connection reduced to about 72% and 57% of those for pure cyclic shear and axial strengths, respectively (see Table 4.6). It shows that for the type C connection, the influence of the cyclic shear forces on the reduction of ultimate cyclic axial strength was more severe than the effect of cyclic axial force on the degradation of ultimate cyclic shear strength. The reduction of the shear strength was the consequence of additional stresses induced at the concrete-steel interface due to the cyclic axial forces, which in turn weakened that region for the succeeding application of the shear forces (the following cycles). The reduction of the cyclic axial strength due to the cyclic shear forces can be explained similarly.

Crack patterns, failure modes and anchor strains

The Type C connection specimens subjected to different combination of T/V ratios (CST1, CST2 and CST3) showed similar crack patterns. The crack patterns for the Specimen CS were basically the same as the crack patterns for the specimens subjected to combined cyclic loading. The main difference was that the crack pattern for Specimen CS was symmetric due to symmetrical reversed cyclic shear loading. However, for the combined reversed cyclic loadings, the reversed loads were not symmetric, i.e. loads were the resultant of the shear and the axial tensile forces in one direction, and the

resultant of the shear and axial compressive forces in the other direction. Therefore under combined cyclic loading, the two symmetrical parts of the connection (right and left sides) did not experience the same induced stresses when the loads were cyclically reversed, which resulted in unsymmetric crack patterns. The crack patterns for the Specimen CT were completely different due to the nature of the loading; the crack patterns were symmetric but not similar to those for the Specimen CS. The final failure modes for the specimens subjected to combined loading (CST1, CST2 and CST3) were similar, and were mostly due to the concrete splitting-spalling along the tensile leg, concrete crushing-spalling surrounding the steel angle and crushing of the concrete between the anchor legs close to the edge (see Figures 4.18(b), 4.24(b) and 4.30(b)). The final mode of failure for the Specimen CS was due to the extensive crushing-spalling of the concrete close to the edge (see Figure 4.8(b)), which was different from those for the combined loading.

Under combined loading, the leg in the tension zone (left side of connection) was subjected to shearing and tensile stresses due to the shear (dowel action) and the axial forces, respectively. This caused high tensile stresses to be induced in the connection and weakened the concrete surrounding the anchors, damaging these locations to develop the failure mechanism.

Energy dissipation and ductility level

Figure 6.13 compares the cumulative energy dissipation versus the ductility level for all of the specimens in Series C (type C connection). The Specimen CS tested under pure cyclic shear forces had the smallest energy dissipation and ductility ratio ($\mu=5.1$), except for the Specimen CT which had a lower ductility ratio ($\mu=4.6$). The Specimen CST2 ($T/V=1.0$) dissipated the largest energy compared with the other specimens in Series C. The energy dissipation for the Specimen CST3 ($T/V=0.3$) was close to that for the Specimen CST1 ($T/V=0.7$). This corresponded to the connection response, since the hysteresis response of the Specimen CST2 exhibited the largest loops compared to the loops for the other specimens, resulting in a larger cumulative dissipated energy. The Specimen CST1 attained the highest ductility ratio ($\mu=6.8$), while the ductility ratios for the Specimens CST2 and CST3 were the same ($\mu=6.0$). Generally, it can be noted for the Type C connection that a higher T/V ratio gives a larger cumulative dissipated energy.

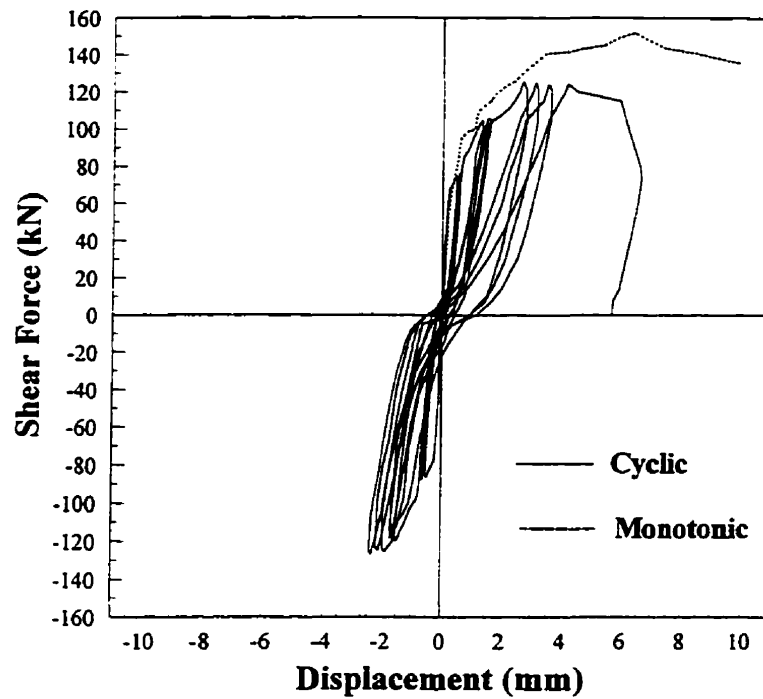


Figure 6.12 Comparison of monotonic and cyclic responses for specimens in Series C (Specimens CS and CMS)

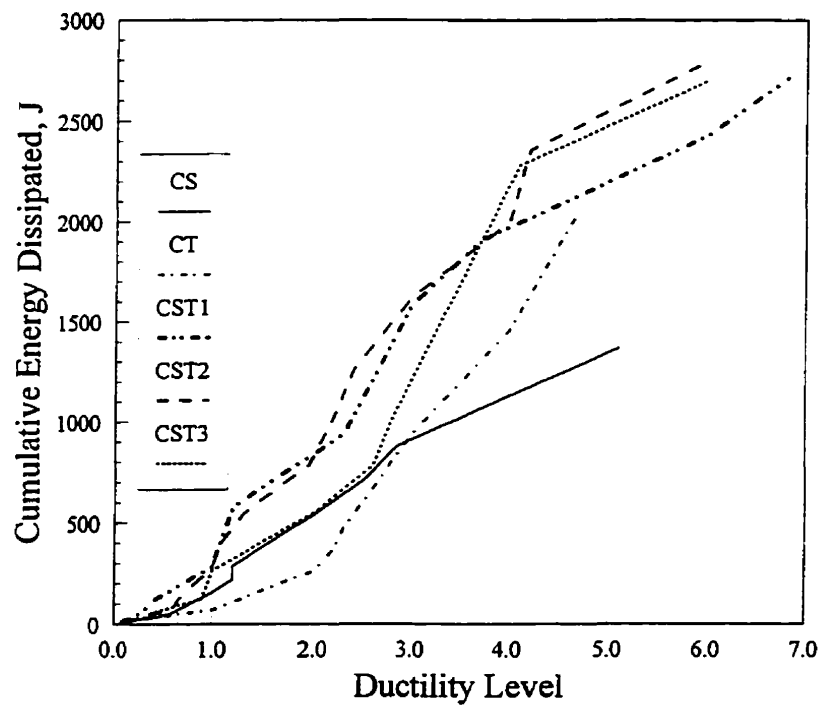


Figure 6.13 Comparison of cumulative dissipated energy for specimens in Series C

6.3.4 Series D Specimens

6.3.4.1 Comparison of Monotonic and Cyclic Shear Responses

Figure 6.14 compares the responses for Type D connection (Specimens DS and DMS) under monotonically increasing and reversed cyclic loads. Figure 6.14 shows that the load-deflection curves remained linear up to a load value 100 kN (Cycle 3A) and coincident to each other for both the monotonic and the cyclic responses. This indicates that the loading cycles in the elastic stage did not decrease the stiffness and strength of the connection. The cracking load for the cyclic loads (65 kN) was smaller than that for monotonic loads (70 kN), see Table 6.5 and Figure 6.14. The lower cracking load for the cyclic loads was due to the propagation and enlarging of the micro-cracks subjected to reversed stresses which were induced in the connection by the reversed cyclic forces. After cracks initiated, the stiffness of the connection did not change noticeably, since the addition orthogonal reinforcing anchor in the connection increased the stiffness of the connection.

At the onset of inelastic behaviour, after extensive cracking (load of 100 kN), the connection stiffness degraded significantly for the cyclic loading, however, it was not noticeable for the monotonic response (Figure 6.14). This was the consequence of crack propagation in the concrete, which weakened the bond strength at the concrete-steel anchor interface due to the reversing stresses induced in the connection by the reversed cyclic loading. The yielding load for the monotonic loading (205 kN) was greater than that for the cyclic loads (150 kN), see Table 6.5. The "tension stiffening" of the concrete between the cracks basically increased the stress required to cause a unit strain in the concrete and steel composite compared with a unit strain in a bare reinforcing bar, thus the yield stress for a bar encased in concrete is larger than that for bare bar. In fact, the reversed cyclic loading reduced the bond strength more than that for the monotonic loads. The cyclic loading reduced the effect of "tension-stiffening" as compared with that for the monotonic loading, therefore the stress required for yield strain under cyclic loading was less than that needed for the yielding of reinforcing anchor when subjected to monotonic loading. Consequently, the yielding strength for the cyclic loading was less than that for the monotonic loading. The Specimens DMS and DS exhibited different crack patterns.

The crack pattern was symmetric and more extensive for the Specimen DS compared to that for the Specimen DMS, which was a consequence of reversed stresses induced in the connection due to the cyclic loading. The Type D connection attained higher ultimate strength value for monotonic loads (226 kN) compared to that for the cyclic loads (190 kN). This shows that the ultimate strength of the connection subjected to reversed cyclic loads reduced to about 84% of that for the monotonic loading, and are presented in Table 6.5 (β value). The ultimate monotonic displacement ($\Delta_u=7.7$ mm) was close to that for the cyclic displacement ($\Delta_u=8.1$ mm), however, the cyclic displacement ductility ratio ($\mu=6.2$) was two times the monotonic displacement ductility ratio ($\mu=3.1$). During application of the monotonic loading, the concrete crushed in front of the steel bar anchor (concrete-bar interface) and it was trapped between the bar and the surrounding concrete, and thus it continued to carry the applied load. During reversed cyclic loading: 1) the crushed concrete at the concrete-steel bar interface did not remain trapped, and 2) upon load reversal the connection which had already cracked was subjected to compressive stresses which were weaker than the uncracked concrete. These phenomena explain the lower strength and higher displacement ductility for reversed cyclic loading.

Although both Specimens DS and DMS failed due to diagonal cracking, the final modes of failure were different; under cyclic loading, the failure occurred along the two diagonal cracks at the end of inclined legs which were connected to each other with a crack forming at the end of orthogonal leg (a cone failure surface), while for the monotonic loading, it was along the diagonal crack at the end of one inclined leg (tensile leg) intersecting the end of the orthogonal leg (See Figures 4.35 (b) and 4.39(b)). Formation of the shear "cone failure surface" for cyclic loading was a consequence of the compression zone being severely cracked due to the reversed stresses in the previous loading cycles in the opposite direction.

Furthermore, the entire length of the steel anchor had yielded at the ultimate load under both cyclic and monotonic loading.

6.3.4.2 Cyclic Response under Combined Shear and Axial Forces

In this section, the overall response of identical specimens in Series D (Type D

connection) subjected to different combinations of reversed cyclic shear and axial (T/V ratios) are discussed. The hysteresis responses, ultimate strengths, displacement ductility levels, and the capability of the specimens to dissipate energy for the different T/V ratios are compared to provide practice-oriented methods to design and analyze such connections when subjected to earthquake loading.

Hysteresis responses , strengths and displacements

The nature of the hysteresis shear response of Specimen DST3 (application of 30% cyclic shear in the direction normal to the shear) was similar to the response under pure cyclic shear forces (see Figures 4.36 and 4.56). This shows that up to a T/V ratio =0.3, the axial load did not change the nature of the hysteresis shear response. However, the ultimate shear strength of the connection decreased to about 87% of that for the pure cyclic shear strength, see Table 4.12. The stiffness of the connection (slope of the load-deflection curve) for the Specimen DST3 (T/V=0.3) decreased more throughout the cycles in comparison with that for the Specimen DS, which was the results of small cyclic axial forces applied along with the cyclic shear loads. The horizontal displacements for the Specimen DST3 were larger than those for the Specimen DS compared at the same load values (see Figures 4.36 and 4.56). This was related to the small applied cyclic axial forces resulting in increased concrete deterioration at a particular load value and in turn increased the horizontal displacement compared with that for the Specimen DS.

Increasing the cyclic axial load to 70% of the shear load for the type D connection (Specimen DST1 ,T/V=0.7), the shear hysteresis response was different from those for the Specimens DS and DST3 (T/V=0.3). This shows that under moderate cyclic axial forces the nature of the response had changed (see Figures 4.36 and 4.44). This was attributed to the additional stress distribution induced in the connection by the cyclic axial forces, which was different from the distributed stresses under the pure cyclic shear loading, thereby influencing the connection response. The stiffness and the strength degradations throughout the load cycles was more significant for the Specimen DST1 (T/V=0.7) than for the Specimens DS and DST3 (T/V=0.3), see Figures 4.44, 4.36 and 4.56, due to the high percentage of applied cyclic axial loads. The nature of the axial hysteresis response for the Specimen DST1 was different from the response of the Specimen DT (pure cyclic

axial loads) (see Figures 4.45 and 4.40). The principal difference was the pinching in the axial response of the Specimen DST1 due to the application of the cyclic shear forces. The displacements for the Specimen DST1 ($T/V=0.7$) were larger than those for the Specimens DS and DST3 ($T/V=0.3$), see Figures 4.44, 4.45, 4.56 and 4.57. The experimental findings show that under combined loading with T/V ratio $=0.7$, the strength of the connection decreased to about 68% and 47% of those for the Specimens DS and DT, respectively, as shown in Table 4.12.

The Specimen DST2 subjected to high percentage of cyclic axial loads ($T/V=1.0$), revealed the nature of responses to be similar to those for the Specimen DST1 ($T/V=0.7$). However, the Specimen DST2 responses was different from the response of the Specimens DST3 and DS. It indicated that the nature of the responses for the connection subjected the combined cyclic forces with T/V ratios between 0.7 and 1.0 were similar. The rate of the shear strength degradation due to the cyclic axial forces for the Specimens DST1 ($T/V=0.7$) and DST2 ($T/V=1.0$) were the same. The rate of the strength degradation for the Specimens DST3 ($T/V=0.3$) and DS also exhibited the same trends, as for the Specimens DST3 ($T/V=0.3$) and DS. The ultimate displacement of the Specimens DST2 was larger than those for Specimens DST1 ($T/V=0.7$) and DST3 ($T/V=0.3$). It shows that with an increase in the T/V ratio (for T/V ratios between 0.7 and 1.0), the ultimate displacement increases. The results show that for the combined loading case with T/V ratio $=1.0$, the strength of the connection reduced to about 55% and 58% of the values for the Specimens DS and DT, respectively (see Table 4.12). As a consequence, the application of cyclic shear force reduced the connection shear strength, as the cyclic axial load degraded the shear strength of the connection. Generally, the decrease of cyclic shear strength was attributed to the extensive stresses induced in the connection (concrete-steel bar interface and the concrete regions) due to influence of the cyclic axial load. This weakened the connection at these particular locations (weakened both the bond between the steel and the concrete and the concrete) for the subsequent cyclic shear forces. The reduction of cyclic axial strength due to the application of the cyclic shear forces applied simultaneously with cyclic axial loads can be explained similarly.

Crack patterns, failure modes and anchor strains

The type D connection subjected to different loading combination i.e. different T/V ratios (Specimens DST1, DST2 and DST3) shows similar cracking patterns consisting of splitting and diagonal cracks. The cracking pattern for the specimen subjected to pure cyclic shear (DS) was basically the same as that for the combined cyclic loads, i.e. consisted of splitting and diagonal cracks. The difference was that the cracking pattern was symmetrical for the Specimen DS because the loads were applied alternately symmetric. The resultant forces under combined loading were not the same when alternately reversed (combination of shear plus tensile axial or shear plus compressive axial), which in turn caused unsymmetrical crack pattern. Major diagonal cracks were formed at the end of the tensile leg and 100 mm away from the leg end for combined cyclic loadings and pure cyclic shear loading, respectively [see Figures 4.39(b), 4.49(b), 4.55(b) and 4.61(b)]. This was due to the principal stresses induced in the connection by the resultant of combined cyclic shear and axial forces. The crack pattern for the Specimen DT was similar to that for the Specimen DS. This was related to the distributed stresses in the connection transferred by the bond strength along the reinforcing bar anchors, which caused diagonal and splitting cracks along the anchors. However, the density of the cracks for the Specimen DS was higher compared with that for the Specimen DT [see Figures 4.39(b) and 4.43(b)].

The final failure modes for the specimens tested under combined cyclic loads were similar; this was due to the failure along the major diagonal crack at the end of the anchor in tension zone [see Figures 4.49(b), 4.55(b) and 4.61(b)]. This was due to extensive tensile stresses induced in that region during the application of the combined shear and tensile axial loads. The final mode of failure for the Specimen DS was similar to those for the combined cyclic loading, the only difference was the location of the failure (Figure 4.39(b) and 4.49). This difference is due to the stresses induced in the connection by the cyclic axial forces. The Specimen DT showed a different mode of failure compared with the other specimens in Series D, which was symmetrical diagonal failure due to the nature of cyclic axial forces (see Figure 4.43(b)). The stresses induced in the two inclined legs were equal and some stress was induced in the orthogonal leg.

This mechanism of load transfer caused symmetric stress distribution, resulting in symmetrical diagonal failure mode (cone surface failure). Furthermore, the reversed cyclic axial loading caused exhaustion of the concrete tensile strength and crushing of the concrete along the major diagonal crack, weakening the region to develop the mechanism of failure.

Energy dissipation and ductility level

Figure 6.15 compares the cumulative energy dissipation versus the ductility level for the specimens in the Series D. The specimen tested under pure cyclic shear loads (Specimen DS) and the specimens subjected to combined cyclic loads (Specimen DST1, DST2 and DST3) showed very similar energy dissipations (see Figure 6.15). This showed that for the specimens in the Series D which exhibited similar mechanism of failure, the energy dissipations were alike. The Specimens DS, DST1 ($T/V=0.7$) and DST2 ($T/V=1.0$) displayed almost equal displacement ductility ratio, as shown in Figure 6.15. The Specimen DT tested under pure cyclic axial loads had the smallest energy dissipation and displacement ductility ratio, except for the Specimen DST3 ($T/V=0.3$) which had the lowest displacement ductility ratio. This is related to development of the failure on the concrete cone surface which was accompanied mostly with the exhaustion of the concrete tensile strength. Typically, this type of load resistance mechanism are associated with small hysteresis loops and low energy dissipation.

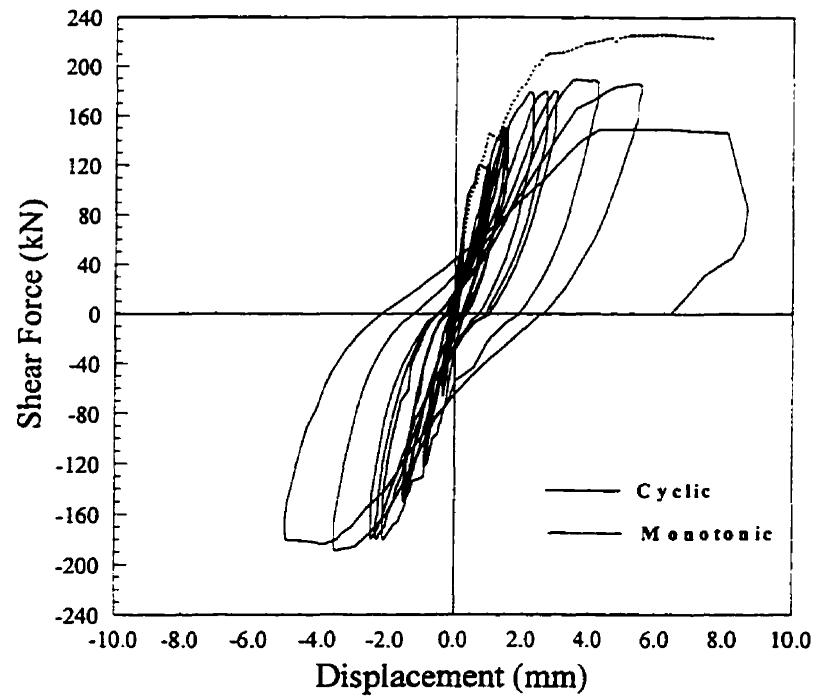


Figure 6.14 Comparison of monotonic and cyclic responses for specimens in Series DS (Specimens DS and DMS)

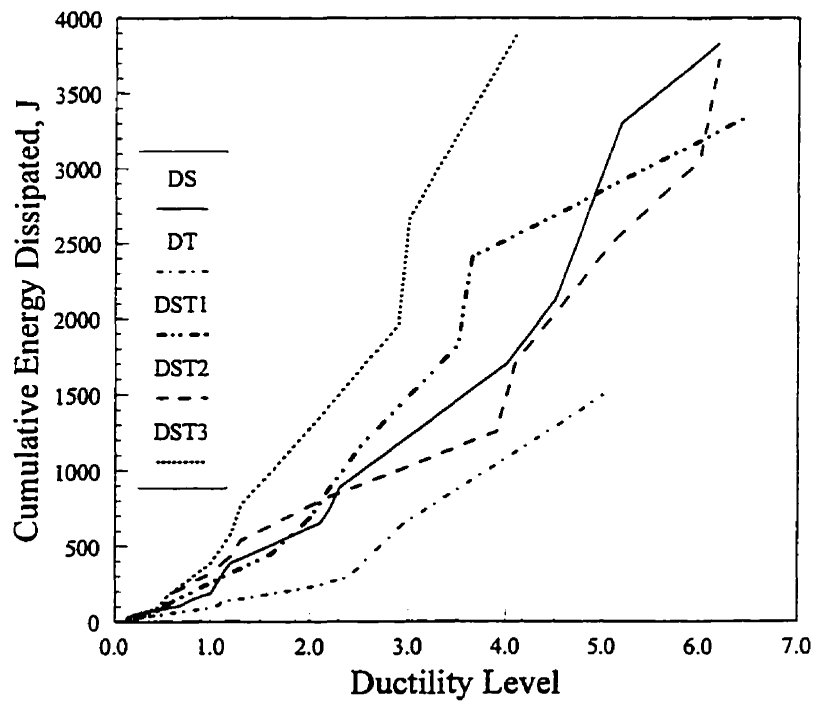


Figure 6.15 Comparison of cumulative dissipated energy for specimens in Series D

6.3.5 Specimens in Series E

6.3.5.1 Comparison of Monotonic and Cyclic Shear Responses

Figure 6.16 compares the responses of type E connections tested under monotonically increasing and reversed cyclic shear forces (Specimens ES and EMS). Up to a load of 15 kN, the load-deflection curve remained linear and coincident with that for the monotonic loads (Figure 6.16). This shows that the loading cycles in the elastic region did not decrease the stiffness and strength of the connection. Once the specimen cracked, the stiffness degradation was more significant for the cyclic loads compared with that for the monotonic response (see Figure 6.16). This was related to the crack propagation in the concrete and the significant increase in the strains in the stud after yielding due to the cyclic loads.

The cracking load for the monotonic loading (29 kN) was larger than that for the cyclic loads (15 kN), see Table 6.5, which was due to the initiation and propagation of micro-cracks under reversed cyclic stresses induced in the connection by the cyclic loads. The yielding strength for the cyclic loading (25 kN) was less than that for the monotonic loading (45 kN). The lower value for cyclic loading was in fact related to extensive concrete crushing in front of the steel plate and spalling of the concrete cover over the stud which resulted in easy movement of the steel plate followed by bending of the studs. This caused significant increase in the strain in the studs, thus the yielding strength of the specimen subjected to cyclic loading decreased in comparison with that for monotonic loading.

The crack pattern differed for the cyclic loads with an additional diagonal crack and more concrete spalling in front of the steel plate in comparison with the monotonic crack pattern. This was due to the sign change (compressive or tensile) and repeating of the induced stresses because of the reversing load cycles, weakening the concrete and influencing the crack pattern.

The cyclic ultimate strength (46 kN) decreased to 86% of the monotonic strength (53 kN), as shown in Table 6.5. This was the consequence of the exhaustion of the concrete strength surrounding the steel plate and development of local failure mechanism in the studs (necking of the studs) under reversed cyclic stresses induced in the

connection by the cyclic loading. In addition, under monotonic loads the stud in compression did not experienced large strains after yielding and its continued contribution to the resistance of the applied load explains the higher strength for the monotonic loading. While under reversed cyclic loads, both studs underwent large strains alternately resulting in a decrease of the shear strength of the steel studs. The ultimate displacement for the monotonic loading (3.4 mm) was close to that for the cyclic loading (3.2 mm), as shown in Figure 6.16.

The final mode of failure was shear fracture of the studs for both the monotonic and the cyclic loading, however, for the cyclic loading, the failure occurred due to fracture of each stud at the peak of positive and negative loads one at a time and was accompanied with more crushing of the concrete around the steel plate in comparison with that for the monotonic loads.

6.3.5.2 Cyclic Response under Combined Shear and Axial Forces

The response parameters for identical specimens from the Type E connection subjected to different T/V ratios, are discussed and compared. This provides research data to establish design criteria and equations for determining the strength of the connection subjected to combined seismic forces.

Hysteretic responses , strengths and displacements

The nature of the shear response under the application of 30% of cyclic shear forces in the normal direction simultaneously with the shear load (Specimen EST3) was close to the response for pure cyclic shear loads (see Figures 5.20 and 5.4). However, the loops were larger for the Specimen EST3 than that for the Specimen ES due to the application of cyclic axial loads. This was related to the application of the small axial force which increased the stresses induced in the connection and the stud anchor due to the shearing forces, which is reflected in the area of the loops in the hysteresis shear response. However, the ultimate shear strength of the connection decreased to about 90% of the pure cyclic shear strength, as shown in Table 5.6. The slope of the shear force-deflection curve (connection stiffness) of the Specimen EST3 reduced more throughout the cycles when compared with that for the Specimen ES, which was a

consequence of the cyclic axial loads causing the connection to be more flexible. The displacements had increased compared with those for pure cyclic shear (see Figures 5.4 and 5.20). This was a consequence of the axial loads, which caused more deterioration in the concrete surrounding the steel plate and increased the strains in the stud anchors, which in turn resulted in an increase in the displacements. The pinching which was severe for the pure cyclic shearing loads was reduced due to the large residual displacement at unloading for the Specimen EST3. Thus the area under the loops increased significantly compared with Specimen ES.

Increasing the percentage of the cyclic axial load to 70% of the cyclic shear force ($T/V=0.7$, Specimen EST1, moderate cyclic axial force), caused the shape of the shear response to be different from that for the Specimen ES. It shows that for T/V ratios larger than 0.3, after the maximum strength is attained, the rate of the strength degradation increased rapidly due to the application of the cyclic axial loads. This was a consequence of the axial stress induced in the studs by the axial forces. Thus the studs were subjected to combined shear and axial stresses which reduced the strength of the connection very significantly (see Figure 5.10). More pinching occurred in the load-deflection curves for the Specimen ES compared with those for the Specimens EST1 and EST2, since the axial load caused larger residual displacement after unloading, thus decreasing the pinching. The nature of the axial hysteresis response was different from that for the Specimen ET (pure cyclic axial loading), see Figures 5.11 and 5.7). This was attributed to the effect of cyclic shear applied simultaneously with cyclic axial loads. The horizontal displacement for the Specimen EST1 was larger than that for the Specimen ES at a similar load value. Comparing the horizontal displacement between the Specimen EST1 ($T/V=0.7$) and the Specimen EST3 ($T/V=0.3$), the Specimen EST3 shows larger horizontal displacement. However, the ultimate axial displacement was larger for Specimen EST1 compared with that for Specimen EST3 due to higher value of the applied cyclic axial loads.

The ultimate strengths under combined loading with T/V ratio =0.7 show that the connection strengths decreased to about 80% and 70% of those for the pure cyclic shear and axial loads, respectively, as shown in Table 5.6.

The Specimen EST2 subjected to combined cyclic shear and axial forces with high T/V ratio =1.0 indicates different hysteresis response than that for the Specimen ES (see Figures 5.4 and 5.15); wider loops for the shear response in comparison with Specimen ES were due to the effect of the high cyclic axial loads which increased the horizontal displacements at same load value. The nature of hysteresis responses for the Specimens EST2 (T/V=1.0) and EST3 (T/V=0.3) were similar (see Figures 5.15 and 5.20), while the hysteresis response for the Specimen EST1 (T/V=0.7) was different than those for the Specimens EST2 (T/V=1.0) and EST3 (T/V=0.3). It can be related to the effects of result of combined shear and axial stresses, which depended on the T/V ratio, changing the stress distribution in the connection. The shape of shear and axial hysteresis responses for the Specimen EST2 were not similar. The main difference was that for axial hysteresis response the loops in negative loading direction were narrower and remained in positive displacements direction compared to those for the shear hysteresis response (see Figures 5.15 and 5.16). The pattern of axial hysteresis response of the Specimen EST2 (T/V=1.0) was not similar to the response of the Specimen ET (pure cyclic axial load), as shown in Figures 5.16 and 5.7. This was attributed to the effects of application of cyclic shear forces on the distribution of stresses in the connection and mainly on the strains in the stud anchors, which influence the axial hysteresis parameters significantly.

The ultimate horizontal displacement for the Specimen EST2 (T/V=1.0) was smaller than those for the Specimens EST1 (T/V=0.7) and EST3 (T/V=0.3), however, the ultimate axial displacement was larger than those for the specimen subjected to smaller T/V ratio (Specimens EST1 and EST3). Comparing the ultimate displacements for the different T/V ratio indicates that with an increase in the percentage the cyclic axial forces (increase in T/V ratios) up to 0.7, the horizontal displacements increased and for larger T/V ratio =0.7, the horizontal displacement decreased. This was attributed to the action of the shear and axial stresses in the stud cross section.

The experimental findings for the combined loading with T/V ratio =1.0 shows that the strength of the connection reduced to about 56% and 78% of those for the pure cyclic shear (ES) and the pure cyclic axial (ET) strengths, respectively. It shows that for the Type E connection, the influence of the axial forces on the degradation of the ultimate

shear strength is more severe than the effect of shear force on the reducing the ultimate axial strength. The reduction of strengths in all different load combinations compared to pure cyclic loading were due to the additional stresses induced in the steel studs and the concrete surrounding the steel plate by the cyclic axial loads, thus weakening the strength at those locations for the succeeding application of the shear forces (following cycles). The same phenomenon explains the degradation of the axial strength due to the shear forces.

Crack patterns, failure modes and anchor strains

Specimens under combined loading (EST1, EST2 and EST3) exhibited similar crack patterns. However, the crack pattern for the specimens under pure cyclic shear and axial loadings were different from those subjected to the combined loads (different T/V ratios). The main difference was that for pure cyclic shear loading, the diagonal and splitting cracks formed, while for the combined loading condition only the concrete cover over the studs spalled off. This was related to the axial stresses induced in the concrete-stud interface, which caused spalling of the concrete. Furthermore, for Specimen ET symmetrical cracks formed at the corners of steel plate, due to the pure cyclic axial stresses induced in the connection. The different crack patterns for the Specimens ES and ET were related to the fact that for pure cyclic shear loading, the stresses in the sides changed alternately from tensile to compressive stresses and vice-versa, while for the pure cyclic axial load, the entire connection was subjected alternate tensile and compressive stresses.

The mode of failure for the combined loading cases and the pure cyclic shear were similar which was due to the fracture of the studs anchor accompanied by the crushing-spalling of the concrete around the steel plate, as shown in Figures 5.6, 5.14(b), 5.19(b) and 5.24(b). The final failure mode of the Specimen ET (pure cyclic axial) was due to the pullout of the stud anchor from the surrounding concrete, which was different from that for the pure cyclic shear and combined loading. For the combined load, the stud anchor in the tension zone was subjected to high cyclic shear and tensile-compressive axial stresses which decreased the shear strength of the studs. Under reversed cyclic loads, this phenomenon induced reversed stresses in the stud cross-section resulting in

stud necking and development of shear fracture failure mechanism.

Energy dissipation and ductility level

Figure 6.17 compares the cumulative energy dissipation versus the ductility level for the specimens in the Series E. The Specimen ES shows the largest cumulative dissipated energy and ductility ratio compared to the other specimens in Series E, except for the Specimen ET. The specimen subjected to T/V ratio =0.3 dissipated more energy and had larger displacement ductility ratios compared with that for the Specimens EST2 (T/V=1.0) and EST1(T/V=0.7). This was related to the hysteresis responses, especially the shear response, and since the hysteresis shear response of the Specimen EST3 consisted of larger loops, this resulted in greater cumulative energy dissipation. The Specimens EST2 (T/V=1.0) and EST1 (T/V=0.7), indicate almost the same capability to dissipate energy, however the ductility level was different, as shown in Figure 6.17. In general, it can be noted that for the Type E connection a higher T/V ratio resulted in a lower energy dissipation. The Specimen ET subjected to pure cyclic axial load dissipated the largest amount of energy and had the longest displacement ductility ratio (see Figure 6.17).

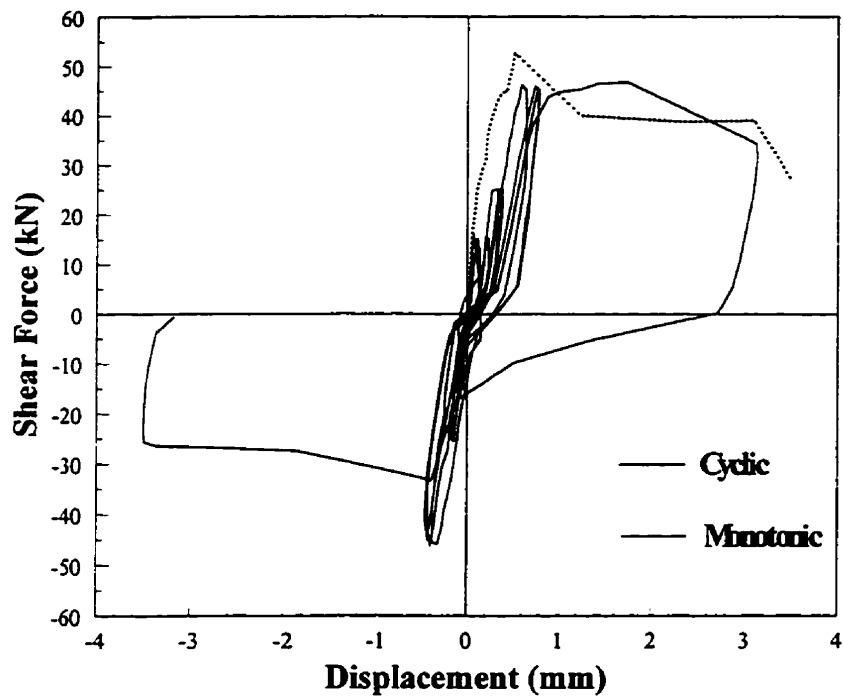


Figure 6.16 Comparison of monotonic and cyclic responses for specimens in Series E (Specimens ES and EMS)

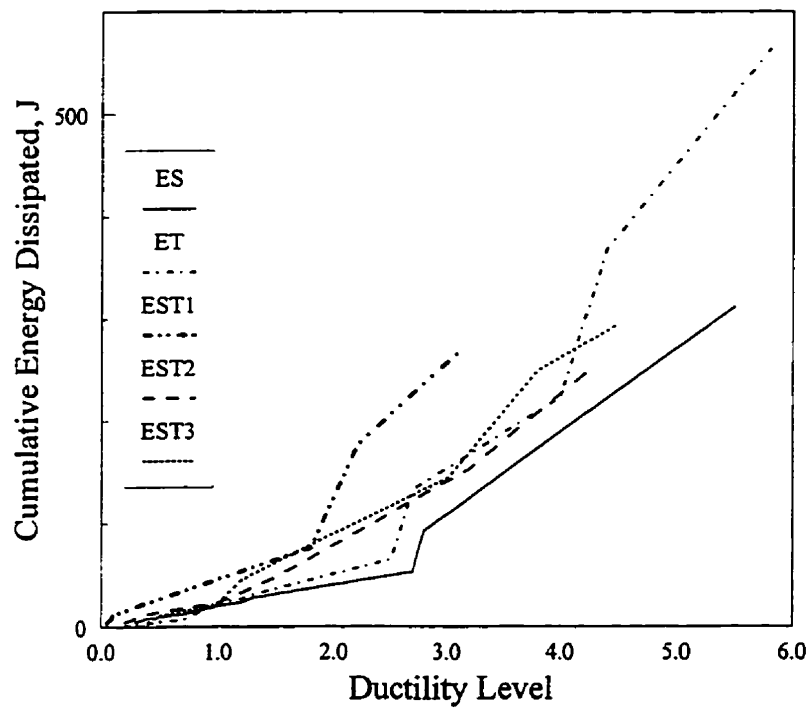


Figure 6.17 Comparison of cumulative dissipated energy for specimens in Series E

Chapter 7

Interaction Relationships

7.1 Introduction

This chapter discusses the development of experimental and design interaction curves and the associated equations for the connections in the Series A through E. The connections of double-tee floor diaphragms and wall systems in precast buildings in regions of high, moderate, or even low seismicity, present a significant seismic risk. However, there is virtually no research data on the seismic resistance of these connections, especially under combined in-plane cyclic forces. To evaluate their seismic response and to develop comprehensive and rational design recommendations and guidelines, it is necessary to understand the performance of the connection subjected to combined cyclic shear and axial forces, which are manifestation of an earthquake loading.

During an earthquake, connections in the precast floor and wall panel systems are subjected to combined seismic in-plane shear and axial forces. A safe and economic design of floor and wall system require an accurate knowledge of the load capacity of the connection under various combinations of shear and axial forces induced during the seismic loading. Therefore, a detailed investigation was undertaken to understand the behaviour of the connections under combined cyclic axial and shear forces and to determine the interaction diagrams and the associated design equations.

7.2 Experimental Interaction Curves

In the previous chapters, the general behaviour and the hysteresis response parameters of the specimens in the Series A through E (connection type A to E), tested under the different combination of in-plane cyclic shear and axial loads, were discussed and compared. Reversed cyclic tests conducted for all series, were preformed with pure shear force, pure axial force, and combined cyclic shear and axial forces for T/V ratios

equal to 0.3, 0.7 and 1.0. The interaction equations for the normalized experimental results were derived for each series; these were detailed in Sections 3.2.7, 3.3.7, 4.2.7, 4.3.7 and 5.2.7 for Series A through E, respectively. Those Sections examined the effect of the cyclic axial forces on the cyclic shear strength, and vice versa. The present section deals with the derivation of the interaction relationships between in-plane seismic forces, when the two extreme values (pure cyclic shear and axial strengths) are incorporated in the interaction equations. The unknown parameters are the maximum in-plane seismic forces induced in the connection in the floor or wall systems, which for a particular precast structure are determined by analyzing the structure subjected to the earthquake or quasi-static loading. These interaction relationships are compared with each other and discussed further.

The experimental results are summarized in Table 7.1, and the maximum cyclic shear strengths, $V_{\text{experiment}}$, were plotted versus the axial strengths, $T_{\text{experiment}}$ for Series A through E in Figure 7.1. The interaction curves were fitted to the experimental findings using the best fit program, and the following interaction equations were derived for the connections Type A through E, and are listed in Table 7.2.

For the connections in Series A:

$$V + \left(\frac{T}{6}\right)^{1.75} \leq 150 \quad (7.1)$$

For the connections in Series B:

$$V + \left(\frac{T}{5}\right)^{1.55} \leq 200 \quad (7.2)$$

For the connections in Series C:

$$V + \left(\frac{T}{14} \right)^{2.1} \leq 125 \quad (7.3)$$

For the connections in Series D:

$$V + \left(\frac{T}{6} \right)^{1.54} \leq 190 \quad (7.4)$$

For the connections in Series E:

$$V + \left(\frac{T}{10.8} \right)^{3.5} \leq 45 \quad (7.5)$$

where V and T are the applied in-plane shear and axial forces, respectively, which are calculated based on the analysis of the structure subjected to the earthquake or quasi-static loading. The material resistance factors must be incorporated in these equations resulting in design interaction equations which can be used for design purposes, see Section 7.3.

Figure 7.1 compares the interaction curves developed for the Connections Type A through E. It is noted from Figure 7.1 that Connections Type B and D display larger strengths under the combined cyclic in-plane forces. This results from the detailing of the 90° standard hooks and an additional orthogonal reinforcing bar in Connections Type B and D, respectively. Connections type A and C show reasonable capacities to resist combined in-plane seismic shear and axial forces. However, the Connection Type E displayed very small strength under the in-plane cyclic forces.

Generally, it is noted from Figure 7.1 that the cyclic shearing capacity of the connection decreases when cyclic axial forces are applied simultaneously. This is due to the fact that the cyclic axial forces added to the cyclic shear forces induce extra stresses at the interface of the concrete and the steel anchors and in the concrete around the steel angle or plate, which results in a reduction of the shear strength. Furthermore, under cyclic load most of the applied forces are carried throughout bond stresses which are

Test types	Ratio of cyclic axial to shear force	Ultimate cyclic strengths (kN)	
		Shear (kN)	Axial (kN)
AS	-	150	-
AST3	0.3	126	30
AST1	0.7	88	62
AST2	1.0	75	75
AT	-	-	105
BS	-	200	-
BST3	0.3	150	41
BST1	0.7	140	80
BST2	1.0	100	95
BT	-	-	150
CS	-	125	-
CST3	0.3	120	30
CST1	0.7	100	65
CST2	1.0	85	85
CT	-	-	145
DS	-	190	-
DST3	0.3	166	30
DST1	0.7	130	51
DST2	1.0	105	63
DT	-	-	180
ES	-	46	-
EST3	0.3	40	13
EST1	0.7	35	22
EST2	1.0	25	25
ET	-	-	32

Table 7.1 Summary of Interaction Test Results for Connection Type A through E

transferred mechanically by bearing of the bar lugs against the surrounding concrete. Therefore, the tensile and the compressive strengths of the concrete, and the cover thickness play a dominant role in controlling the connection behaviour for this loading case. Because the thickness of the concrete panel was small, a tensile splitting mechanism along with lower bond stresses was dominant. This effect was more severe with extra reversed cyclic axial loads since (1) more severe cracking occurred, and (2) the

load repetitions increased the energy available for crack propagation. The cyclic axial strength decreases when cyclic shear forces are applied simultaneously. However, Figure 7.1 shows that for the connections Type C and E, if an axial force less than 30% of the shear force is applied simultaneously in the axial direction, the shear strength of the connections does not decrease significantly.

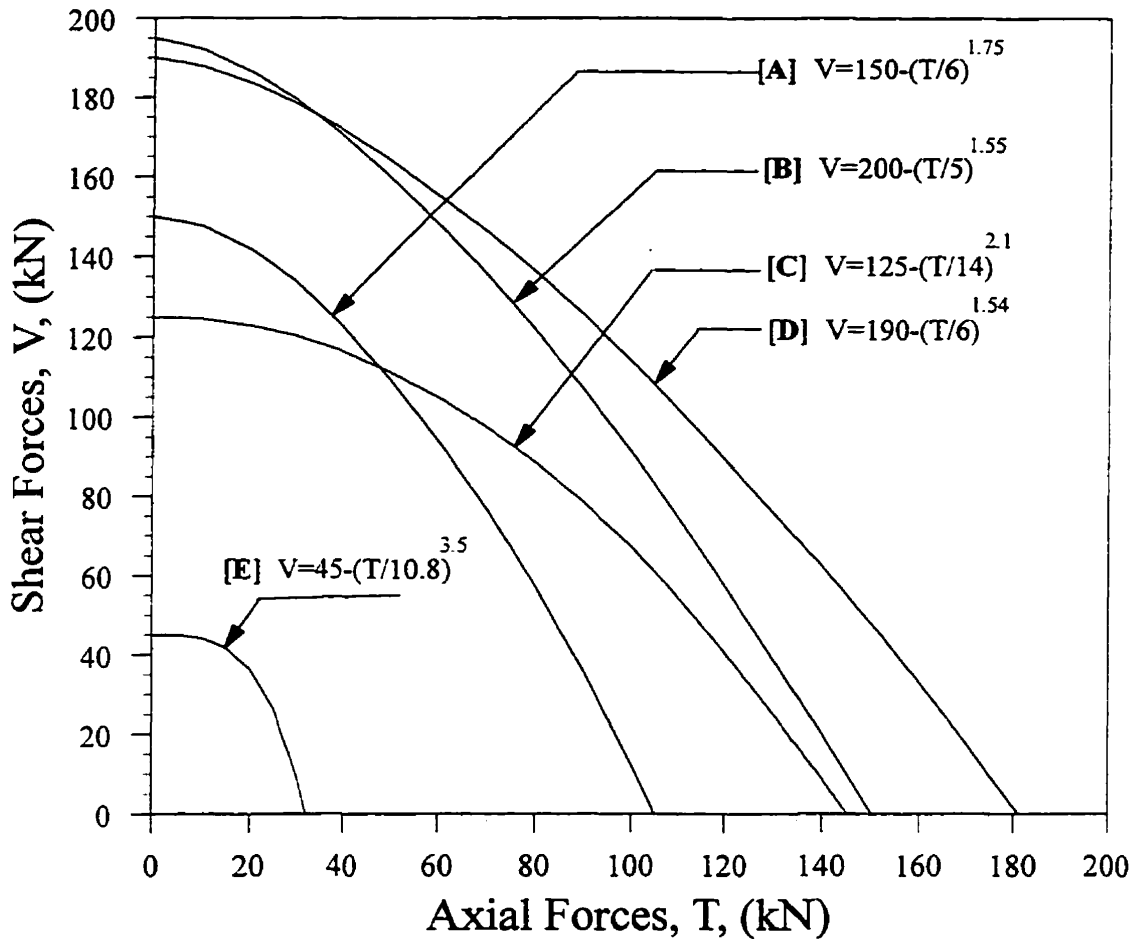


Figure 7.1 Experimental interaction diagrams for Connection Type A through E

7.3 Design Interaction Curves

7.3.1 Factored Resistance

The CSA Standard CAN3-A23.3-M94 adopted a set of load factors, which are used to factor the loads used in the analysis of connections in precast concrete structures. These load factors are accompanied by the resistance factors applied to the material strengths to achieve the resistance level of safety, where the resistance factors are applied to material strengths (to allow for approximations in the calculations and variations in the material strengths, workmanship and dimensions) rather than to the nominal resistance, for example, ϕV_n and ϕT_n .

The Clauses 8.3 and 16.1 of the CSA Standard CAN3-A23.3-M94 give values of ϕ_c and ϕ_s for the concrete and the reinforcing steel, respectively. The factored concrete strengths used in checking the ultimate limit states are taken as $\phi_c f'_c$ or $\phi_c \sqrt{f'_c}$ and for reinforcing bars it is taken as $\phi_s A_s f_s$, where $\phi_c=0.65$ and $\phi_s=0.85$.

For all type of the connections tested under reversed cyclic loads, the ultimate strengths of the connections were governed by the concrete failure (i.e. cracking and crushing of concrete or/and splitting and pull-out bond failures), except for the specimens in Series E where the ultimate capacities were controlled with fracture of the stud steel. Therefore, regardless of the details of the equations which should be used to calculate the strength of the connections, it is clear that for the connections in the Series A to D, the tensile-compressive (axial) strengths, shear strength and the combined shear-tensile strength are functions of either f'_c or $\sqrt{f'_c}$, (i.e. T or V is a function of f'_c or $\sqrt{f'_c}$), justifying the use of factored resistance given by $\phi_c f'_c$ or/and $\phi_c \sqrt{f'_c}$. On the other hand, for design purposes, the experimental strengths V_o (the maximum force that the connection can take in pure shear), and T_o (the maximum force that the connection can resist in pure tension), must be substituted by $\phi_c V_o$ and $\phi_c T_o$, respectively, to achieve the normalized design interaction equations. For the specimens in Series E because the failure was due to the fracture of steel, the strengths are function of $A_s f_s$, therefore the factored resistance would be given by $\phi_s A_s f_s$. Therefore, the terms V_o and T_o in Series E should be replaced with $\phi_s V_o$ and $\phi_s T_o$, respectively to achieve the normalized interaction design equations.

Following the same logical reasoning and procedure to derive the design interaction equations, the experimental ultimate strengths for the connections subjected to different combination of cyclic loads must be substituted by their factored strengths (i.e. strengths must be calculated using the factors ϕ_c or ϕ_s as applicable). On the other hand, for design purposes the experimental interaction relationships V and T should be substituted by V_ϕ/ϕ_c and T_ϕ/ϕ_c , respectively, for Series A to D, to achieve the design interaction equations. To derive the design interaction equation for Series E, the terms V and T needed to be substituted with V_ϕ/ϕ_s and T_ϕ/ϕ_s , respectively.

7.3.2 Design Interaction Equation

Normalized interaction relationships of the form

$$\left(\frac{V}{V_o}\right) + \left(\frac{T}{T_o}\right)^\beta \leq 1.0 \quad (7.6)$$

were developed from the experimental results of the specimens tested under combined cyclic shear and tensile loads (Chapters 3, 4 and 5). Here, β is a variable depending on the type of connections, computed from curve fitting of the experimental ultimate strengths and has values of 1.75, 1.5, 2.0, 1.54, and 3.5 for connections in Series A, B, C, D, and E, respectively. Also, V is the applied shear force, V_o is the maximum force that the connection can take in pure shear, T is the applied force in tension-compression (axial force), and T_o is the maximum force that the connection can take in pure tension-compression (axial force). The experimental interaction equations were derived and discussed in Section 7.2.

The interaction equations for the design of the connections and their normalized forms are derived based on considerations discussed in Section 7.3.1 and are presented in Tables 7.2 and 7.3, along with their simplified versions for use in the design office. In these tables, V_ϕ and T_ϕ are the factored shear and tensile-compression (axial force) forces induced in the connection during an earthquake, respectively. V_o and T_o are the maximum forces that the connection can resist in pure shear and in pure tension-compression (axial force), respectively.

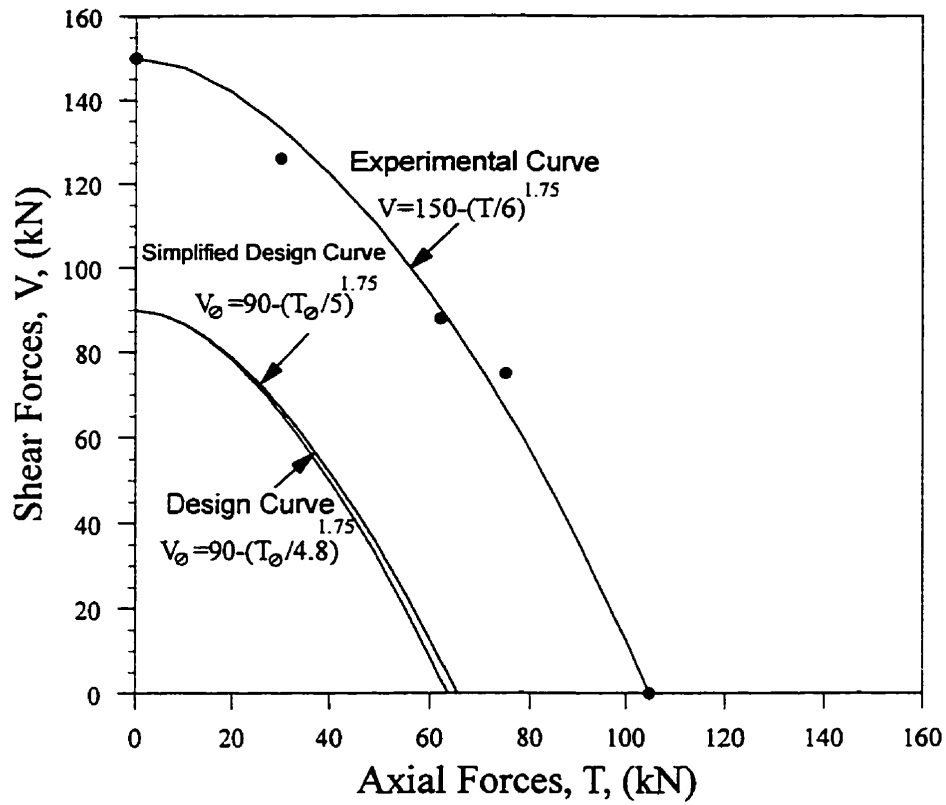
Figures 7.2 to 7.6 show the experimental and the purposed design interaction curves for connections in Series A, B, C, D and E, respectively. The interaction and normalized interaction curves for each series in the related figures are distinguished using the symbols (a) and (b), respectively.

Connection Type	Experimental Equation (derived) [kN]	Design Equation (derived) [kN]	Simplified Design Equation [kN]
A	$V + \left(\frac{T}{6}\right)^{1.75} \leq 150$	$V_{\phi} + \left(\frac{T_{\phi}}{4.8}\right)^{1.75} \leq 90$	$V_{\phi} + \left(\frac{T_{\phi}}{5}\right)^{1.75} \leq 90$
B	$V + \left(\frac{T}{5}\right)^{1.55} \leq 200$	$V_{\phi} + \left(\frac{T_{\phi}}{4}\right)^{1.55} \leq 120$	$V_{\phi} + \left(\frac{T_{\phi}}{4}\right)^{1.5} \leq 120$
C	$V + \left(\frac{T}{14}\right)^{2.1} \leq 125$	$V_{\phi} + \left(\frac{T_{\phi}}{11}\right)^{2.1} \leq 75$	$V_{\phi} + \left(\frac{T_{\phi}}{11}\right)^2 \leq 75$
D	$V + \left(\frac{T}{6}\right)^{1.54} \leq 190$	$V_{\phi} + \left(\frac{T_{\phi}}{5}\right)^{1.54} \leq 114$	$V_{\phi} + \left(\frac{T_{\phi}}{5}\right)^{1.5} \leq 114$
E	$V + \left(\frac{T}{10.8}\right)^{3.5} \leq 45$	$V_{\phi} + \left(\frac{T_{\phi}}{9.6}\right)^{3.5} \leq 38$	$V_{\phi} + \left(\frac{T_{\phi}}{10}\right)^{3.5} \leq 38$

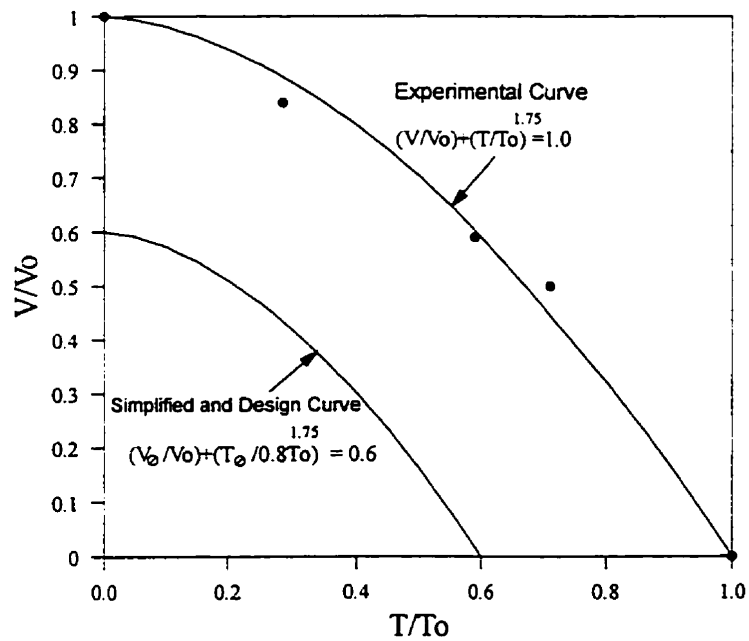
Table 7.2 Summary of Experimental, Design and Simplified Design Version Equations for Connections Type A through E

Connection Type	Normalized Experimental Equation (derived)	Normalized Design Equation (derived)	Simplified Design Version of Normalized Equation
A	$\left(\frac{V}{V_o}\right) + \left(\frac{T}{T_o}\right)^{1.75} \leq 1.0$	$\left(\frac{V_\phi}{V_o}\right) + \left(\frac{T_\phi}{0.8T_o}\right)^{1.75} \leq 0.6$	$\left(\frac{V_\phi}{V_o}\right) + \left(\frac{T_\phi}{0.8T_o}\right)^{1.75} \leq 0.6$
B	$\left(\frac{V}{V_o}\right) + \left(\frac{T}{T_o}\right)^{1.55} \leq 1.0$	$\left(\frac{V_\phi}{V_o}\right) + \left(\frac{T_\phi}{0.84T_o}\right)^{1.55} \leq 0.6$	$\left(\frac{V_\phi}{V_o}\right) + \left(\frac{T_\phi}{0.8T_o}\right)^{1.5} \leq 0.6$
C	$\left(\frac{V}{V_o}\right) + \left(\frac{T}{T_o}\right)^{2.1} \leq 1.0$	$\left(\frac{V_\phi}{V_o}\right) + \left(\frac{T_\phi}{0.77T_o}\right)^{2.1} \leq 0.6$	$\left(\frac{V_\phi}{V_o}\right) + \left(\frac{T_\phi}{0.8T_o}\right)^2 \leq 0.6$
D	$\left(\frac{V}{V_o}\right) + \left(\frac{T}{T_o}\right)^{1.54} \leq 1.0$	$\left(\frac{V_\phi}{V_o}\right) + \left(\frac{T_\phi}{0.84T_o}\right)^{1.54} \leq 0.6$	$\left(\frac{V_\phi}{V_o}\right) + \left(\frac{T_\phi}{0.8T_o}\right)^{1.5} \leq 0.6$
E	$\left(\frac{V}{V_o}\right) + \left(\frac{T}{T_o}\right)^{3.5} \leq 1.0$	$\left(\frac{V_\phi}{V_o}\right) + \left(\frac{T_\phi}{0.89T_o}\right)^{3.5} \leq 0.85$	$\left(\frac{V_\phi}{V_o}\right) + \left(\frac{T_\phi}{0.9T_o}\right)^{3.5} \leq 0.85$

Table 7.3 Summary of Experimental, Design and Simplified Design Version of Normalized Equations for Connections Type A through E

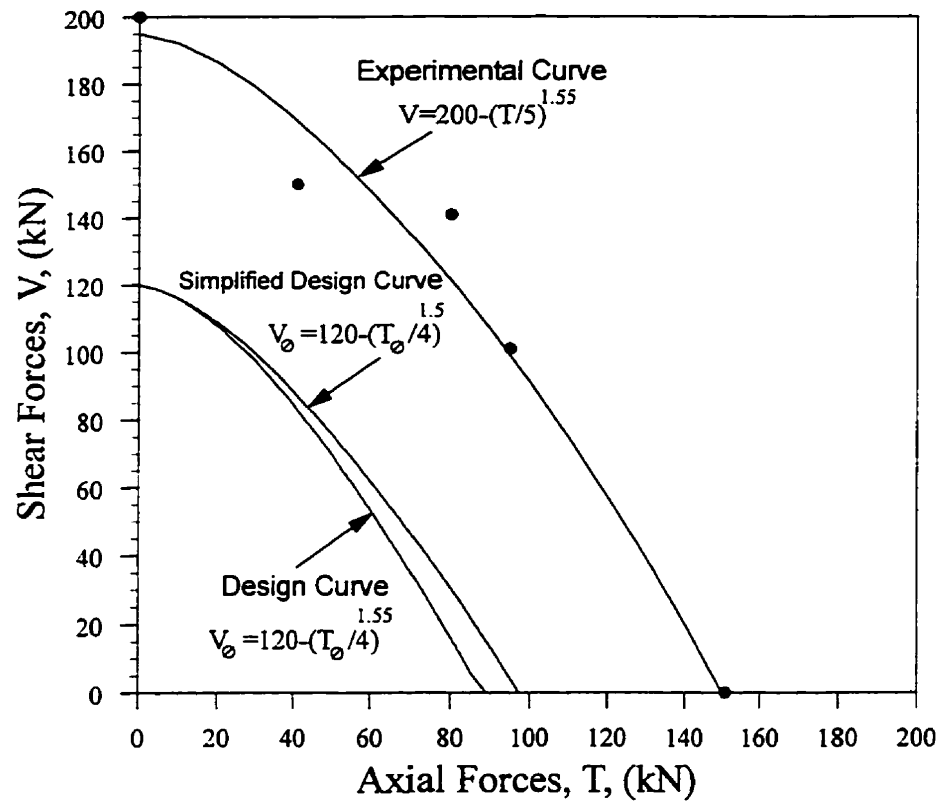


(a) Interaction curves

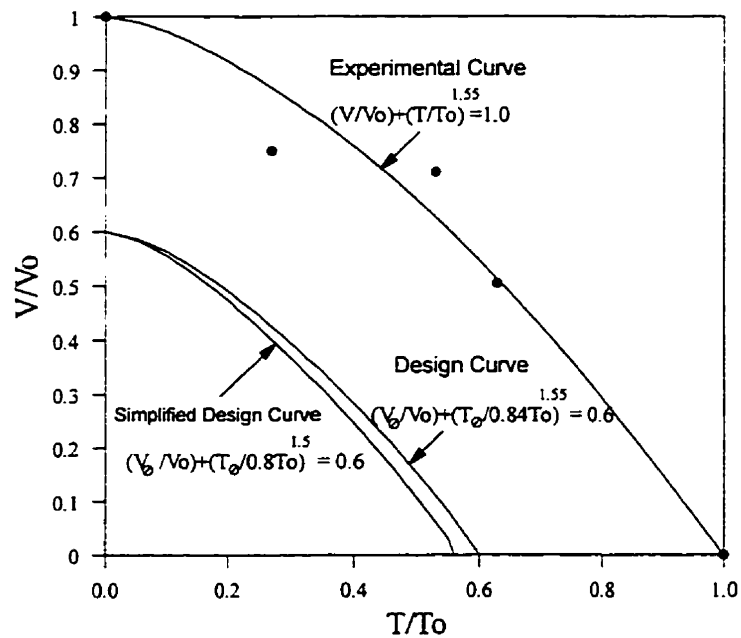


(b) Normalized interaction curves

Figure 7.2 Experimental, design and simplified design interaction diagrams for Connection type A

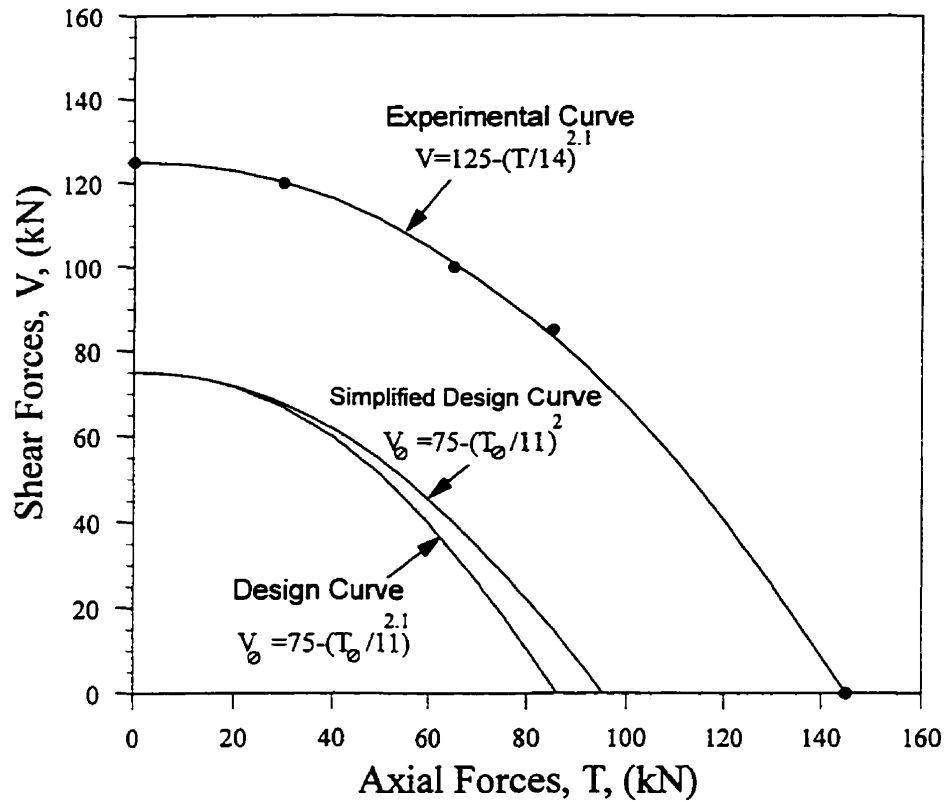


(a) Interaction curves

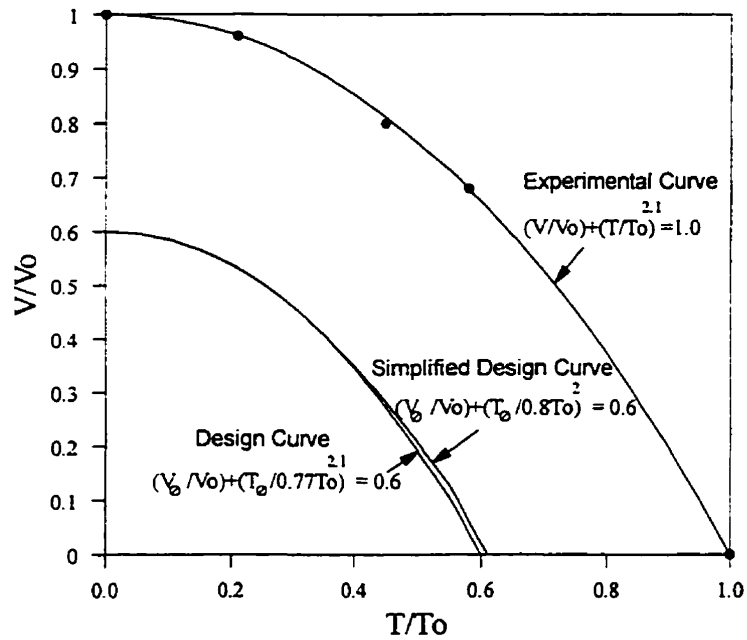


(b) Normalized interaction curves

Figure 7.3 Experimental, design and simplified design interaction diagrams for Connection type B

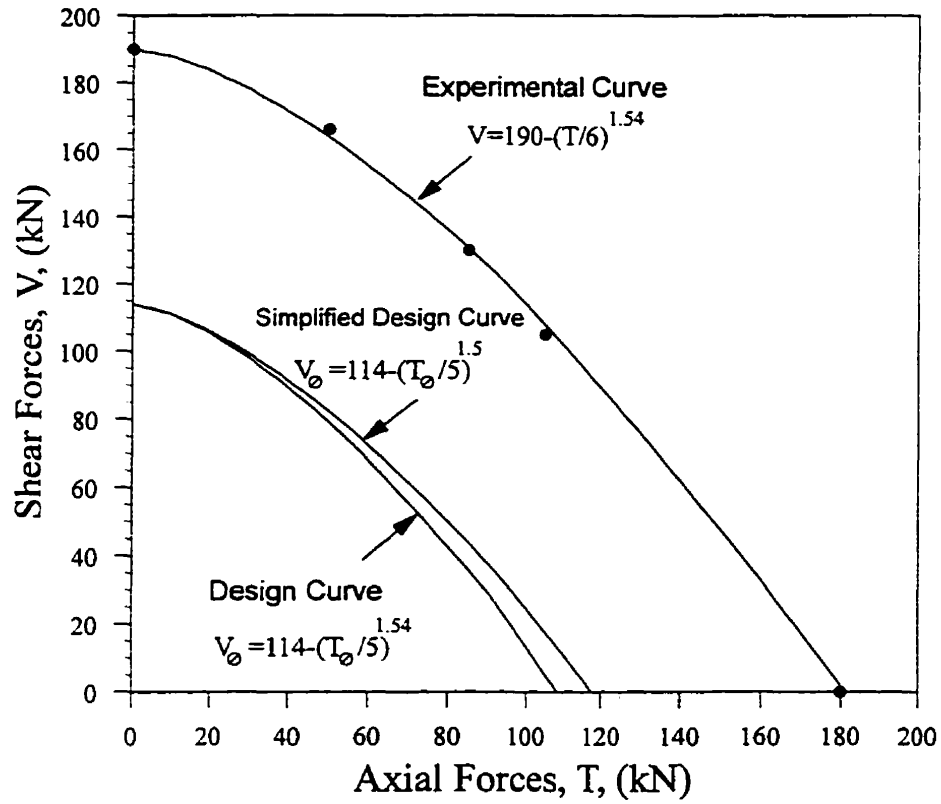


(a) Interaction curves

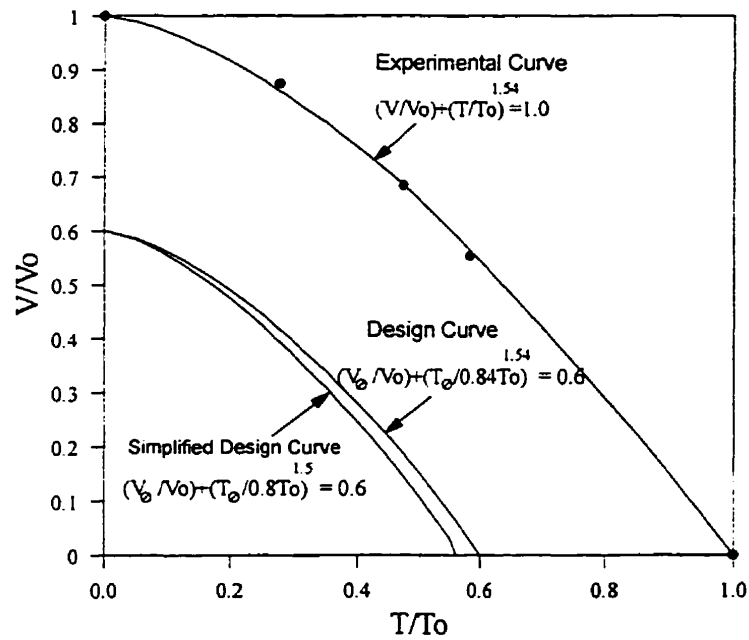


(b) Normalized interaction curves

Figure 7.4 Experimental, design and simplified design interaction diagrams for Connection type C

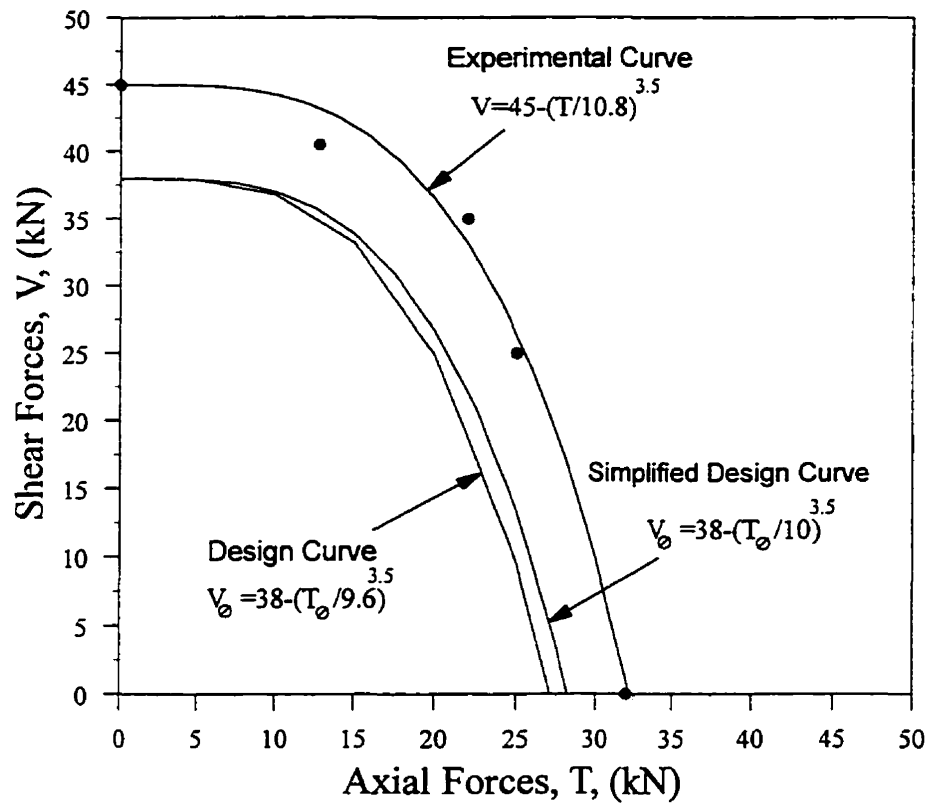


(a) Interaction curves

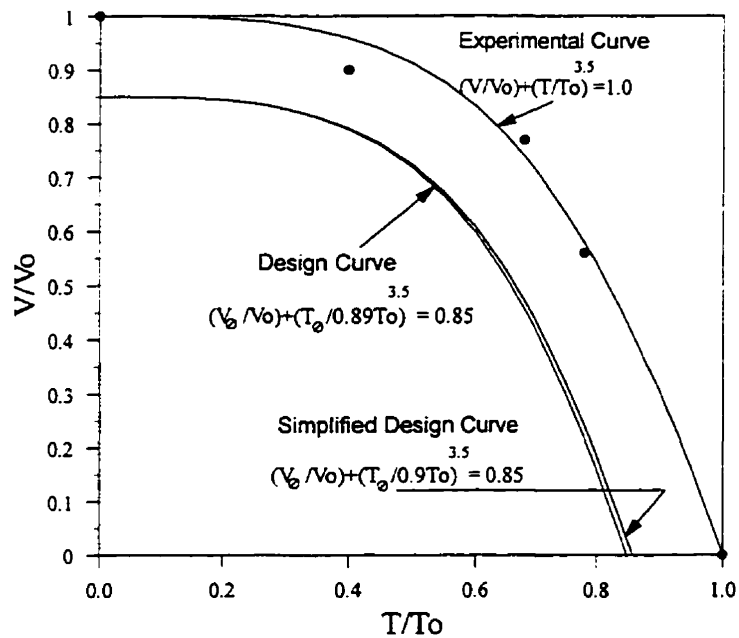


(b) Normalized interaction curves

Figure 7.5 Experimental, design and simplified design interaction diagrams for Connection type D



(a) Interaction curves



(b) Normalized interaction curves

Figure 7.6 Experimental, design and simplified design interaction diagrams for Connection type E

7.3.3 Discussion and Comparison of Design Interaction Relationships

Figure 7.7 compares the simplified design interaction curves developed for the five connections in this research program (connections type A to E). In order to achieve the best performance for the connection subjected to combined cyclic in-plane forces, the connection must be proportioned with a capacity to resist cyclic shear as well as cyclic axial loads. Figure 7.7 reveals this relationship between the cyclic in-plane shear and axial strengths for the connections, i.e. the connections are strong for both reversed cyclic shear and axial forces. The material reduction factors $\phi_c=0.6$ and $\phi_s=0.85$ have been included in developing the equations so that the charts give the factored cyclic shear and axial load V_ϕ and T_ϕ . All of the interaction curves have been based on a yield strength of 400 MPa for reinforcing bars and concrete strengths between 35 MPa and 40 MPa. Appropriate adjustment can be made in deriving interaction curves for other steel and concrete strengths.

The capacity of the connections are illustrated and distinguished with their related interaction diagrams, as shown in Figure 7.7, and the strength of each connection can be determined with the help of its specific interaction diagram. As can be noted from Figure 7.7 and the findings in Chapter 6, the Connections Type B and D show excellent strength and deformability under combined cyclic in-plane shear and axial forces, and therefore their use is recommended for use in areas prone to severe and moderate earthquakes. Connections Type A and C displayed good ultimate cyclic capacities and reasonable deformability and they are recommended for use in regions with moderate and low earthquakes. However, Connection Type E can be utilized in the zones with low seismicity.

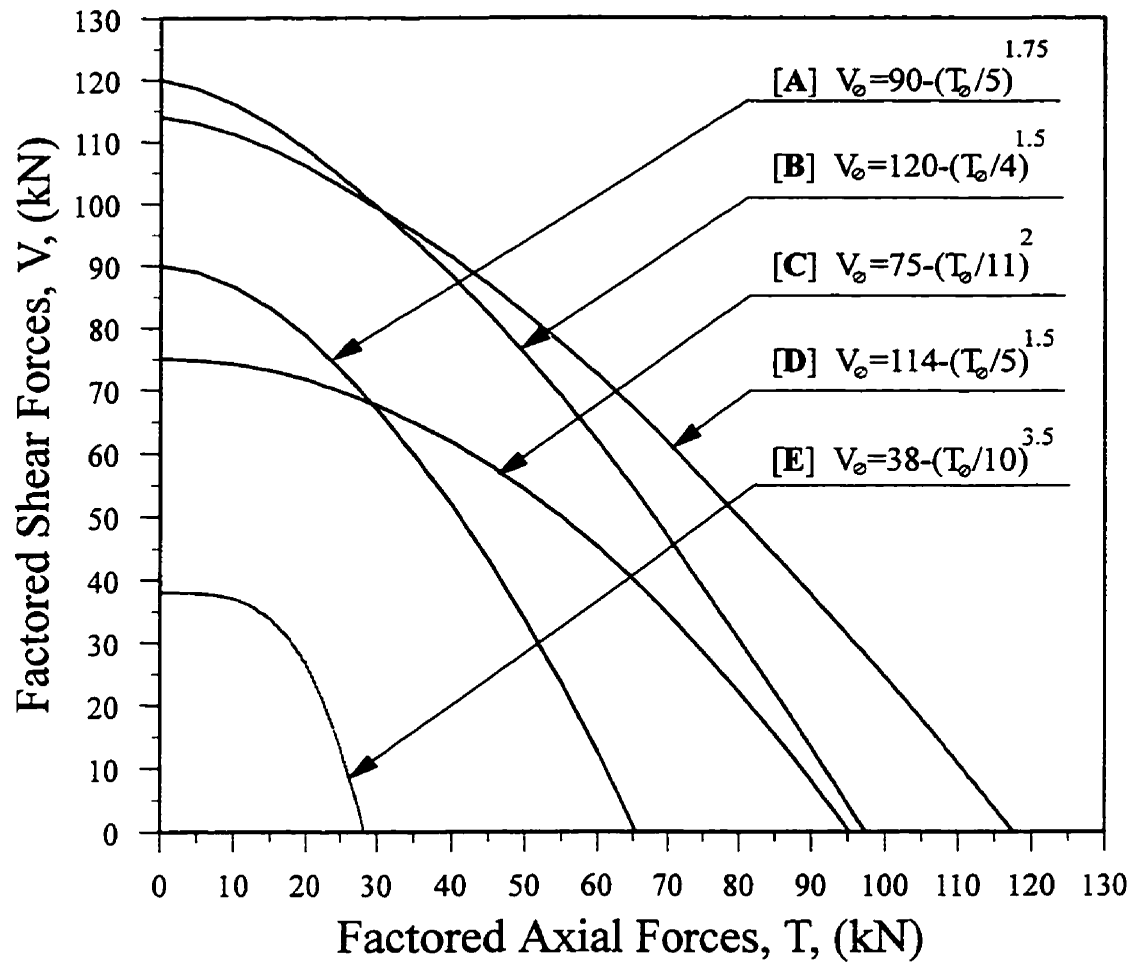


Figure 7.7 Simplified design interaction diagrams for Connection Type A through E

Chapter 8

Comparison of Analytical and Experimental Results

This chapter presents briefly the conceptual methods of analysis developed in this study along with the predicted responses of the five connections subjected to monotonically increasing loads, calculated using the NONLACS program (Ghoneim, 1987, and Razaqpour 1988). The complete monotonic response characteristics under monotonically increasing loads for the shear loading were discussed. The ultimate strength of the connections calculated based on the proposed methods are compared with the experimental data and the NONLACS program results. The connections are also analyzed subjected to different combinations of in-plane shear and axial loads to derive the analytical interaction equations for monotonically increasing in-plane forces. It should be noted that attempt is made to analyze the experimental results for the monotonic shear loading test, which are compared with selected experimental data. However, the analytical interaction curves and the equations are derived from the results of computer analysis of the connections subjected to monotonically increasing combined in-plane shear and axial loads. Also, the computed analytical results are compared with the experimental data for the specimens subjected to cyclic loads to establish the modified analytical curves for design purposes.

8. 1 Design Strength of Connections

On the basis of the resisting mechanism observed experimentally and the physical models describing the ultimate behaviour of the connections, equations are developed for shear and axial strength of the connections under monotonically increasing actions. The pull-out strength in the tensile leg with respect to concrete cover is calculated using the equation proposed by Jimenez (1979), and the length of reinforcing bar is selected based on the anchorage lengths required.

8.1.1 Connection Shear Strength

8.1.1.1 Connection Type A

Based on the experimental observation at the ultimate loading stage (before the connection fails), the concrete bearing against the partial length of the anchor bar at the end of the steel angle in the compression area gets pushed off at the ultimate load. Until this stage, the leg that is in compression is not subjected to a significant stress (the measured strain in the strain gauges during the testing confirms this). The force, F_b , required to fail this concrete is given by

$$F_b = d_b l_e f'_c \quad (8.1)$$

where d_b is the bar diameter, l_e is the effective length of the compressive leg bearing against the concrete, found from the experimental observation to be approximately $19d_b$.

The ultimate tensile strength of the leg is based on the bond capacity of M15 deformed steel bars corresponding to a splitting type of failure. The ultimate tensile strength, the anchorage length and the concrete cover are related as follows:

$$T_u = \frac{d_b l_d C \sqrt{f'_c}}{424 d_b + 6.5 l_d} \quad (8.2)$$

where C and l_d are concrete cover thickness and development length of the anchor bar, respectively.

The contribution of the steel mesh in the tension zone can be determined as following:

$$V_{sh} = n A_{sh} f_{yh} \quad (8.3)$$

where n , A_{sh} and f_{yh} are the number of the steel mesh wires crossing the crack, the area of the steel wire and the yield strength of steel wire, respectively.

Using the equilibrium of the forces in the connection, the ultimate shear strength of the connection, V_{ult} , is given by

$$V_{ult} = (F_b + T_u) \cos 45^\circ + V_{sh} \quad (8.4)$$

The shear strength of connection AMS is $V_{ult} = 185.7 \text{ kN}$ (details of the analysis are given in Appendix A), which is in excellent agreement with the experimental strength (187 kN).

Therefore the connection capacity is predicted excellently by the proposed method.

8.1.1.2 Connection Type B

The mechanism of failure for the Specimen BMS was identical to that for Specimen AMS. In addition, the concrete in front of the standard hook gets crushed, therefore this resistance should be added to the strength of the connection. This resistance can be found approximately by the fact that the distribution of the stresses in front of the hook is a quarter ellipse with the maximum value equal to the concrete compressive strength. Therefore, the contribution of the hook to the connection strength, F_{hook} is:

$$F_{hook} = \frac{1}{4} \pi d_b l_h f'_c \quad (8.5)$$

where $l_h = 12d_b$ is the length of the right angle hook part

The ultimate tensile strength of the hooked bar corresponding to a splitting type of failure is given by:

$$T_u = \frac{d_b l_d C \sqrt{f'_c}}{424 d_b + 6.5 l_d} \quad (8.2)$$

where the development length l_d is equal to $l_{hb} = l_{hd} + R_{ba} + d_b$ and R_{ba} is the inside radius of the hook.

The force required to push off the concrete in the compression bar leg is calculated using equation (8.1). The effective bar length bearing against the concrete was noted to be about $17d_b$, which gives the strength F_b as:

$$F_b = d_b l_e f'_c \quad (8.1)$$

Contribution of the steel mesh in the tension zone can be determined using equation (8.3) as follows:

$$V_{sh} = n A_{sh} f_{yh} \quad (8.3)$$

where the various terms have been explained previously.

Writing the equation for equilibrium of forces for the connection, the ultimate strength of Specimen BMS, V_{ult} , is given by

$$V_{ult} = (F_{hook} + T_u + F_b) \cos 45^\circ + V_{sh} \quad (8.6)$$

The detail of the analysis, as presented in Appendix A, which gives the shear strength of the connection as $V_{ult} = 234.7$ kN. The experimental ultimate strength of the connection is 242 kN, therefore the connection strength is estimated excellently by the proposed method (experimental strength is just about 3 percent more than the predicted strength).

8.1.1.3 Connection Type C

Based on the experimental observation, the concrete along a given length of the bar crushed resulting in the connection failure. In fact, the compressive stresses induced in the concrete due to the bearing against the bars reached the maximum concrete compressive strength. Therefore, the lateral bearing of the bar anchor which act on this length as a stud anchor, controls the resisting mechanism. The bearing resistance of the bar anchor is:

$$F_b = 1.4 d_b l_e f'_c \quad (8.7)$$

It is shown that $5d_b \leq l_e \leq 8d_b$ (Marcus 1951). For Connection Type C, l_e was noted to be about $5.5d_b$. The shear strength of the Connection Type C with two anchor bars is given by:

$$V_{ult} = 2 \times F_b \quad (8.8)$$

The ultimate shear strength of the Connection Type C (details of the analysis are given in Appendix A) is $V_{ult} = 149.8$ kN. The experimental ultimate shear strength of the connection CMS is 152 kN, thus the connection strength is estimated excellently by the proposed method.

8.1.1.4 Connection Type D

Based on the experimental results, the connection failed along the diagonal crack formed in the tension zone. The following phenomena contribute to the ultimate strength

of the connection: strength of the diagonal shear anchor bar crossing the diagonal crack, V_{dh} , shear strength due to the horizontal steel mesh crossing the diagonal crack, V_{sh} , and the strength contribution of the shear resistance of the concrete surrounding the connection anchors, V_{ch} . The ultimate strength of the connection, V_{ult} , is calculated as:

$$V_{ult} = V_{dh} + V_{sh} + V_{ch} \quad (8.9)$$

The strength due to tensile anchor bar, V_{dh} , is given by:

$$V_{dh} = A_{sd} f_{yd} \cos 45^\circ \quad (8.10)$$

where A_{sd} and f_{yd} are the cross-sectional area and the yield strength of the diagonal bar, respectively.

The contribution of the steel mesh, V_{sh} , can be found using equation 8.3 as follows:

$$V_{sh} = n A_{sh} f_{yh} \quad (8.3)$$

where the terms in the equation have been explained before.

The contribution of the concrete to the shear resistance, V_{ch} which act as the "shear-cone" can be calculated as following:

$$V_{ch} = 0.3 A \sqrt{f'_c} \quad (8.11)$$

where A is the effective stress area.

The experimental ultimate strength for the Specimen DMS is 226 kN, therefore the predicted connection strength by proposed method ($V_{ult} = 221.3$ kN; details of the analysis are given in Appendix A) is in excellent agreement with the experimental results.

8.1.1.5 Connection Type E

Based on the failure mode of the connection, which was the shear fracture of the studs, the ultimate strength of the connection was attained when the stress in the studs reached its ultimate strength. Therefore, the ultimate tensile strength should be utilized instead of the yield strength, when the shear friction concept is used to predict the ultimate strength of the connection.

The ultimate shear strength can be calculated as followings:

$$V_{ult} = A_{vf} f_{ult} \mu \quad (8.12)$$

where A_{vf} = cross sectional area of the studs

μ = 0.8 λ for contact plane a full plate thickness below the concrete surface

λ = 1 for normal density concrete

f_{ult} = ultimate strength of the studs

The experimental ultimate strength for connection EMS is 52.6 kN. Therefore the strength of the connection predicted by the proposed method ($V_{ult} = 49.4$ kN; the details of the analysis are presented in Appendix A) is in excellent agreement with the experimental result.

8.1.2 Connection Pull-Out Strength

8.1.2.1 Connection Type A

Since the failure mode along both legs was due to the splitting-spalling, the ultimate strength of the connection should be calculated based on splitting bond strength. The ultimate tensile strengths of both tensile legs are identical, and can be found using equation (8.2) as follows:

$$T_u = \frac{d_b l_d C \sqrt{f'_c}}{424 d_b + 6.5 l_d} \quad (8.2)$$

Writing the force equilibrium equation for the joint using the "truss analogy", the ultimate axial (pull-out) strength of the connection, T_{ult} , is given by:

$$T_{ult} = 2 T_u \cos 45^\circ \quad (8.13)$$

The axial strength of Connection Type A (details of the analysis are given in Appendix B), T_{ult} , is 110.2 kN. The experimental ultimate strength of the connection AT under cyclic loading is 106 kN. Therefore, the proposed method can predict the ultimate capacity of the connection very closely.

8.1.2.2 Connection Type B

After the bond splitting along the legs, there is no significant contribution due to

the bond strength along the legs. Therefore, the entire tensile load is resisted by the concrete in front of the 90° standard hooks and with the steel mesh crossing the splitting cracks. In fact, the ultimate strength is achieved, when the concrete in front of the hooks pushed off by crushing, and the steel mesh crossing the cracks yielded.

The contribution of each hook in the connection strength, T_{hook} is as follows:

$$T_{hook} = \frac{\pi}{4} d_b l_h f'_c \quad (8.14)$$

where $l_h = 12d_b$ is the length of the right angle part of the hook.

Contribution of the steel wire in the tension zone can be determined as follows:

$$T_{sv} = n A_{sv} f_{yv} \quad (8.15)$$

where n , A_{sv} and f_{yv} are number of the steel wires crossing the cracks, the cross-sectional area of the steel wire and the yield strength of the steel wire, respectively.

Writing the equilibrium of the forces in the axial direction for both hooks and the resistance due to the steel mesh, the ultimate strength of Specimen BT, T_{ult} , is given by

$$T_{ult} = 2 \times T_{hook} \times \cos 45^\circ + T_{sv} \quad (8.16)$$

The predicted ultimate strength using the proposed method is $T_{ult} = 157.4 \text{ kN}$ (details of the analysis are given in Appendix B), which agrees very well with the experimental ultimate strength of the Specimen BT (150 kN).

8.1.2.3 Connection Type C

According to the experimental results, the connection failed due to the complete deterioration of the concrete surrounding both anchor legs. Therefore, the splitting bond strength along both anchor bars determines the ultimate strength of the connection under axial load. Regarding the bond strength corresponding to this mechanism of failure, equation (8.2) is used to determine the ultimate strength for each leg, as follows:

$$T_u = \frac{d_b l_d C \sqrt{f'_c}}{424 d_b + 6.5 l_d} \quad (8.2)$$

There is no contribution of the steel mesh, since there is no steel mesh resisting the axial stresses in the cracked region. Therefore, the ultimate axial strength of the Connection Type C, with two anchor bars can be taken as:

$$T_{ult} = 2T_u \quad (8.17)$$

The predicted ultimate strength using the proposed method is $T_{ult} = 162.6 \text{ kN}$. However, the experimental ultimate strength of the Specimen CT is 150 kN, which is 8% lower than the analytical value $T_{ult} = 162.6 \text{ kN}$ (details of the analysis are given in Appendix B), which is due to the cyclic axial loading associated with the spalling of the concrete.

8.1.2.4 Connection Type D

On the basis of the experimental results, the failure mechanism was due to failure along the diagonal crack. As discussed in Chapter 6, the strength of the connection can be determined utilizing the concept of the tensile "shear-cone" failure and accounting for the steel mesh crossing the conical surface (diagonal cracks formed at the ends of all three bars).

The contribution of the concrete to the pull-out resistance, T_{cv} which acts as "shear-cone" can be calculated as follows:

$$T_{cv} = 0.3 A \sqrt{f'_c} \quad (8.18)$$

where A is the effective stress area.

Contribution of the steel mesh crossing the conical surface can be determined using equation (8.15):

$$T_{sv} = n A_{vh} f_{yv} \quad (8.15)$$

where the terms have been explained earlier.

Writing the equation of the force equilibrium in the axial direction for the "shear-cone" resistance and the resistance of the steel mesh, the ultimate strength of Specimen DT, T_{ult} , is given by

$$T_{ult} = T_{cv} + T_{sv} \quad (8.19)$$

The ultimate experimental strength is 175.6 kN, which is just about 2% higher than the predicted value using the proposed method ($T_{ult} = \underline{175.6 \text{ kN}}$; details of the analysis are given in Appendix B), showing excellent agreement. Therefore, the strength of the connection can be found using the proposed method present herein.

8.1.2.5 Connection Type E

Specimen ET failed due to the pull-out of the stud anchors. Therefore, The ultimate strength of the connection is dependent on the tensile strength of the concrete which participates in the failure mechanism. The equation to determine the ultimate strength of the connection can be expressed in terms of the defined concrete cylindrical surface, P times L , and the tensile strength of the concrete, f'_c . Then the ultimate tensile strength of the Connection Type E, T_{ult} , can be calculated by the following equation:

$$T_{ult} = n P l f'_c \quad (8.20)$$

where $P = \pi d_h$ = perimeter of the stud head

l = embedment length

n = number of studs

f'_c = concrete tensile strength

The experimental ultimate strength is 32 kN, which is smaller than the predicted ultimate strength using the proposed method is $T_{ult} = \underline{38.3 \text{ kN}}$ (details of the analysis are given in Appendix B) due to the influence of cyclic loading. However, the method presented herein can be used to found the nominal strength of the connection with reasonable accuracy.

Summary

The analytical ultimate strengths (shear and pull-out) calculated by the proposed methods give very good to excellent agreement with the experimental results for all connection types. Therefore, the theoretical strength of the connections under static loads can be found accurately using the methods proposed in this section which are derived based directly on the observed failure mechanisms.

8.2 Finite Element Analysis

The NONLACS computer program (NON-Linear Analysis of Concrete and Steel structures) can be used to analyze and to trace the non-linear behaviour, cracking and failure modes of reinforced and prestressed concrete beams, shear panels, slabs, folded plates, shells, box girder bridges, and any other spatial structure normally considered to be an assemblage of "thin" plates subjected to bending and in-plane forces.

The results of analyses performed based on the development in the original version of the NONLACS program (FERLACS, by Ghoneim, 1978) are quite interesting and valuable. Comparison of the analytical results obtained by Razaqpour et al (1988) with the test results of the slab-on-girder bridges with composite steel sections establishes the reliability of the NONLACS program in predicting the nonlinear response of complex structures. The results showed that analytical approach using the NONLACS program is a useful tool to predict the response of the prestressed concrete box girder bridges (Razaqpour et al, 1989). The recent research at McGill University by Risha (1991), Shariatmadar and Mirza (1992), Bello (1992), Kheyroddin and Mirza (1994, 1995), Kheyroddin (1996), Shayanfar and Mirza (1995), Manatakos (1996) indicates that the analytical results using the NONLACS program agreed well with the experimental findings for the load-deflection responses, cracking patterns and the failure mode at the ultimate capacity for the R C panel, beams and structural walls and connections in precast concrete structures.

8.2.1 Finite Element Modelling

The NONLACS program employs three types of finite elements:

- (a) A quadrilateral thin shell elements or quadrilateral facet shell elements (QFSE),

Figure 8.1, which is composed of

- i) quadrilateral membrane element, with four nodes, and three degrees of freedom per node, which are two in-plane displacements, u and v , and one rotation normal to plane, ω , and
- ii) bending elements, with four nodes and three degrees of freedom per node, namely, lateral displacement, w and normal rotations θ_x and θ_y . The

program, provides the option of using a cubic displacement field in both directions, representing the general behaviour, or a linear displacement field in one direction and a cubic displacement field in the other direction which best represents the beam behaviour problem.

- (b) One-dimensional uniaxial member, bar elements, with a linear displacement field, which are useful for truss members, prestressing tendons and reinforcing bars.
- (c) A spring element (boundary element) used to model the prescribed displacements at the boundary, elastic supports, and skewed boundary conditions. The element is also used to calculate the reactions at the supports.

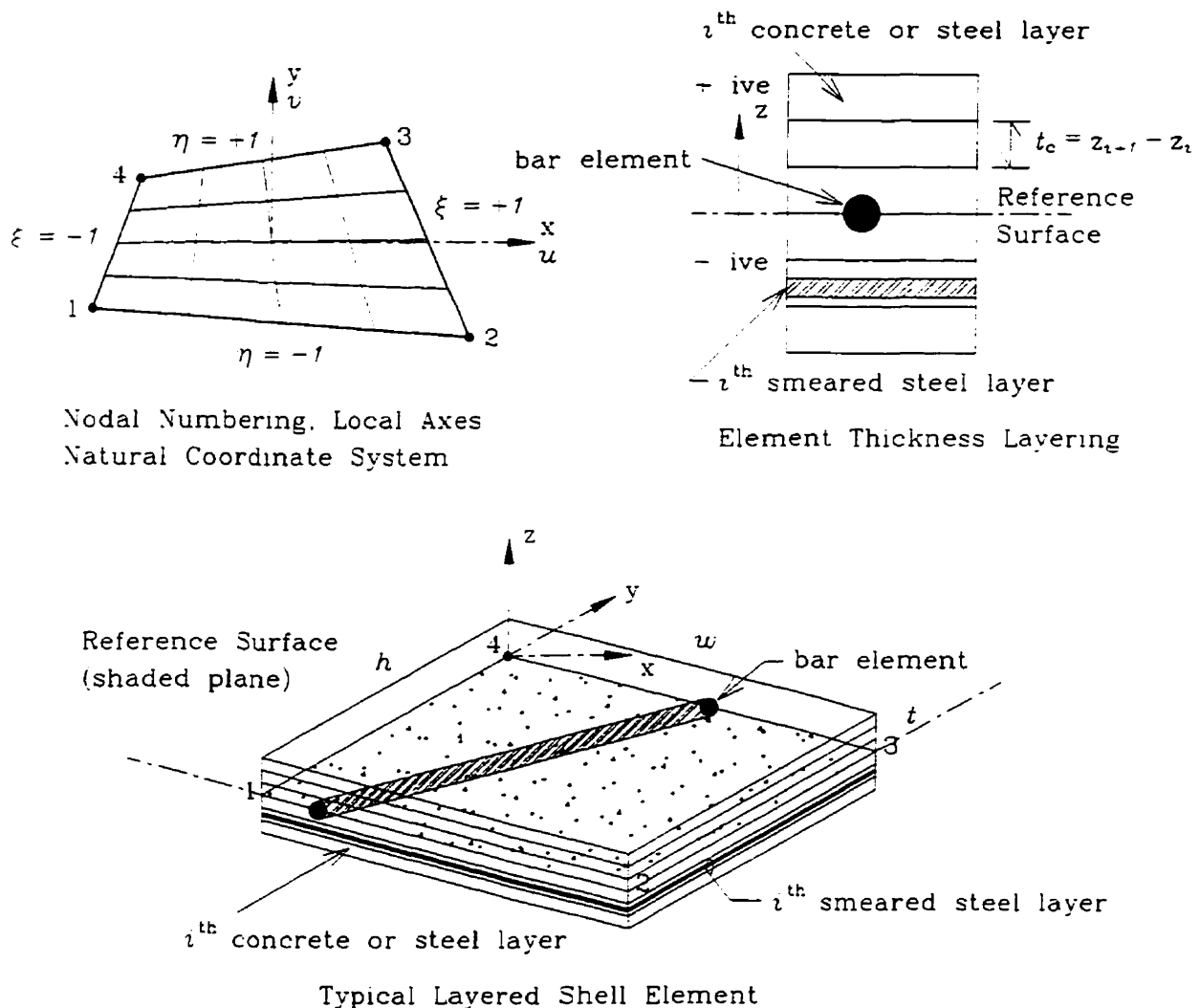


Figure 8.1 Typical facet shell element available in NONLACS program (Ghoneim, 1978)

The structure is divided into an assemblage of some or all of the above element types. A reinforced concrete member is modelled using the layered technique. Each shell or plate bending element is divided into a number of imaginary concrete layers and equivalent "smeared" reinforcing steel layers (Figure. 8.2). Each layer may have different properties and thickness corresponding to its state of stress or strain. The stiffness matrix is calculated by using the numerical integration procedure.

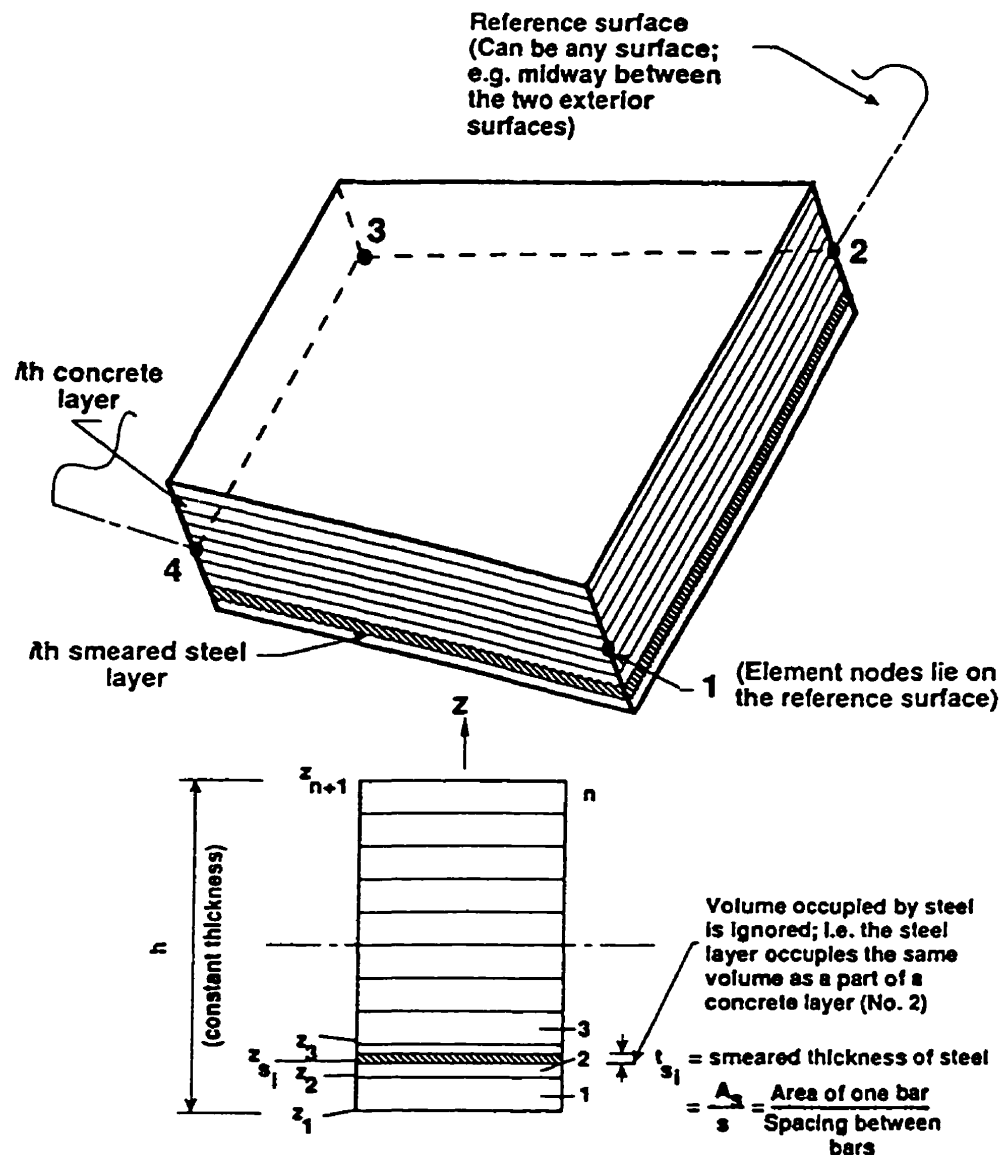


Figure 8.2 Layered element used in the analysis (NONLACS program, Ghoneim, 1978)

Nine Gauss integration points (3x3) are used in the formulation of each element. The layered material properties could be changed from one Gauss integration point to another. The contributions to the element stiffness matrix at the integration points for composite concrete-steel and different elements are obtained by the superposition of the various layer stiffness. Modelling the distributed steel such as reinforcing mesh as equivalent smeared layers is only suitable when steel is fairly distributed over the area of the elements. Concentrated reinforcing or prestressing steel bars are, therefore, modelled as bar elements, assuming perfect bond between the reinforcing steel and the surrounding concrete. However, these bars should be located on the reference surface.

8.2.2 Nonlinear Analysis Method

The NONLACS computer program has the capability of analyzing a linear problem by imposing a one load stage iteration assuming a linear stress - strain relationship. For material constitutive modelling, concrete is idealized as a nonlinear material in compression including strain - softening after attaining the maximum stress and as a linear material in tension with linear unloading characteristics after cracking. In this respect, the NONLACS program uses a uniaxial compressive stress - strain curve. The intrinsic shape consists of two parts [Figure 8.3(a)]. Part I up to a maximum compressive stress is based on Saenz's equation (1964), represented by the equation:

$$\sigma = \frac{E_o \epsilon}{1 + \left(\frac{E_o}{E_{sc}} - 2\right) \frac{\epsilon}{\epsilon_{cu}} + \left(\frac{\epsilon}{\epsilon_{cu}}\right)^2} \quad (8.21)$$

where E_{sc} = secant modulus of elasticity at peak stress

E_o = initial tangent modulus

σ = stress

ϵ = strain

ϵ_{cu} = strain at peak stress.

Part II representing the inelastic strain softening follows the Smith - Young model according to the expression:

$$\sigma = f'_c \left(\frac{\epsilon}{\epsilon_{cu}} \right) e^{\left[1 - \left(\frac{\epsilon}{\epsilon_{cu}} \right) \right]} \quad (8.22)$$

where f'_c is compressive strength, and e is base of natural logarithm.

For uniaxial tensile stress - strain behaviour, the model proposed by Kabir (1976) is used. A linear descending branch for the unloading of the concrete in tension is considered to account for the tension - stiffening after cracking, see Figure 8.3(a).

The concrete strength under biaxial compression and combined compression and tension is determined from the failure envelope of Kupfer and Gerstle (1973). Furthermore the equivalent strain concept due to Darwin and Pecknold (1977) is utilized to relate the increments of stress and strain in the principal directions. Therefore if σ_1 and σ_2 (major and minor principal stresses) exceed the concrete strength calculated using the envelope due to Kupfer and Gerstle, the concrete is assumed to fail along that principal direction. On the other hand, if the equivalent compressive strain in one direction exceeds the maximum concrete strain, ϵ_{cmax} [Figure 8.3(a)] the concrete gets crushed. The cracking of the concrete is determined by checking the magnitudes of σ_1 and σ_2 . When the principal tensile stress at a point (usually a Gauss integration point) equals or exceeds the tensile strength of the concrete, cracking is assumed to occur in a direction perpendicular to the principal tensile stress direction. The effect of the crack is smeared within the element by modifying the constitutive matrix. After cracking, β , the shear retention factor, with a value less than unity, is used to account for the shear stiffness due to the aggregate interlock and dowel action [Figure 8.4].

Normal reinforcing bars and prestressing tendons and bars are modeled as an elasto-plastic strain hardening material. The bilinear stress-strain relationship in Figure 8.3(b) is used to idealize the behaviour of the reinforcing steel under static or cyclic loading conditions. As can be seen from Figure 8.3(b), in the case of loading, and reloading, the Baughinger effect, or load reversal which reduces the yield strength of steel, is also considered. The material parameters, f_y , E_s , E_s'' , and ϵ_{su} , which are respectively, the yield stress, the modulus of elasticity, the strain hardening modulus and the ultimate strain need to be input to define the stress-strain relationship. For shell plate elements,

the model adopted in this program is based on the so-called incremental theory of plasticity in which the total plastic strain is obtained by scanning plastic strain increments.

8.2.3 Analysis Procedure

For nonlinear analysis, an incremental iterative tangent stiffness technique, with the load applied in appropriate increments, is used to trace the nonlinear response of the structure. During the first iteration after the application of a load increment, the incremental displacements are calculated using the tangent stiffness evaluated at the end of the previous load increment. The incremental stresses and strains in the concrete and the steel are calculated and added to the previous total stresses and strains to obtain the current total stresses and strains. This is traced for each integration point for all of the elements. Use of the nonlinear constitutive relations gives the "real" stresses related to the "current" strains. Any unbalanced stresses due to the difference between the "true" and the "approximated" stresses are calculated which are used to compute the equivalent nodal forces. These forces are imposed at the beginning of the second iteration. The procedure is continued until convergence is achieved, or the maximum number of iterations specified by the user in the data is reached. The solution is stopped when the divergence criterion, identified by large unbalanced forces or displacement increments, occurs. This is considered as failure or instability of the structure

Program Limitations

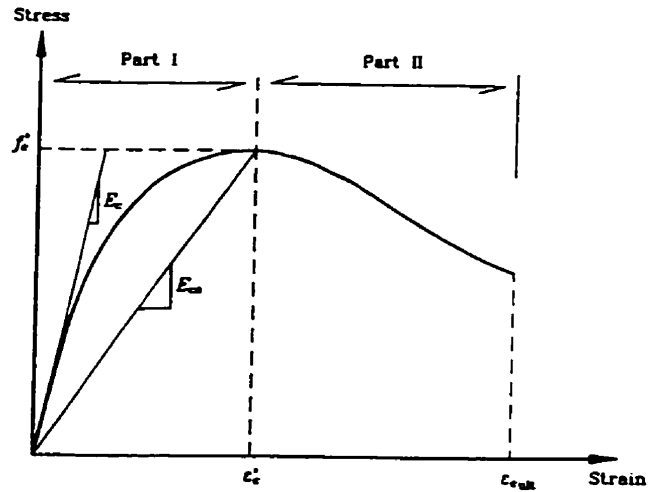
The NONLACS program does not consider the following:

1. Large displacement or geometrical nonlinearity;
2. Shear deformation normal to the surface, i.e. punching shear;
3. Bond slip at the interface of the concrete and the steel reinforcement,
4. Beam elements

Also, the program does not include a graphics module to plot the stresses and deformations in the structure. Therefore, a post-processor program (Quick Basic program) was developed to sort the load-deflection and load- steel bar strain data for a specified node and to plot the undeformed and deformed structure and to scheme the crack patterns for a prescribed load step.

• Uniaxial Compressive σ - ϵ Curve

- Part I - Initial Portion upto $f'_c \Rightarrow$ Saenz (1964)
- Part II - Strain-Softening Portion $\Rightarrow$ Smith-Young (1955)



• Uniaxial Tensile σ - ϵ Curve

- Tension-Stiffening Portion $\Rightarrow$ Kabir (1976)

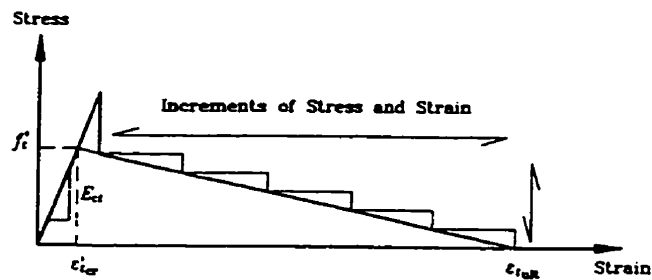


Figure 8.3 (a) Compressive and tensile stress-strain relationship for concrete used in NONLACS program (Ghoneim, 1978)

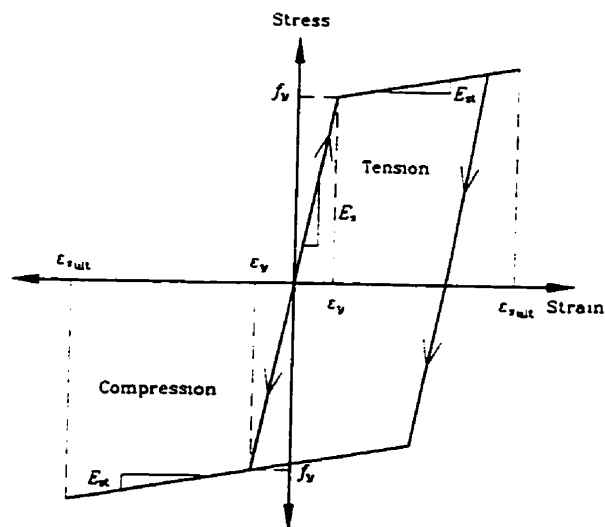
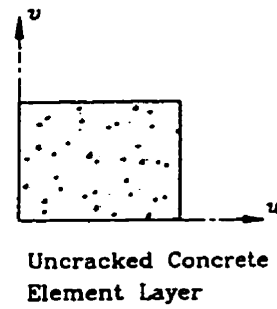


Figure 8.3 (b) Stress-strain relationship for steel (NONLACS program, Ghoneim, 1978)

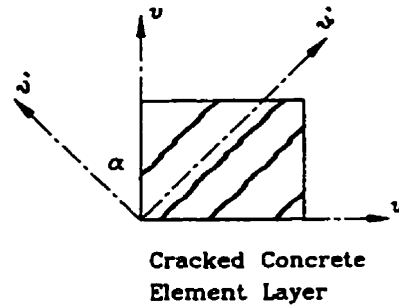
Uncracked Concrete

- Tangent moduli E_1 and E_2 coincide with principal stress directions σ_1 major and σ_2 minor



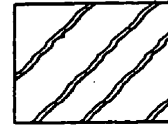
Cracked Concrete - Tension Stiffening Model

- Principal direction - Normal and parallel to cracks
- Zero stiffness perpendicular to crack
- Condition σ_1 or $\sigma_2 \geq f'_t \Rightarrow$ Cracked

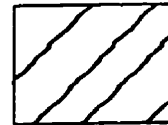


- FIRST Crack: $\sigma_1 \geq f'_t \Rightarrow E_1 = 0$

$$[D] = \begin{bmatrix} 0 & 0 & 0 \\ 0 & E_2 & 0 \\ 0 & 0 & \beta G \end{bmatrix}$$



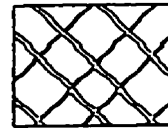
First Crack Open



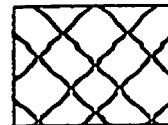
First Crack Closed

- SECOND Crack perpendicular to σ_1 : $\sigma_1 \geq f'_t \Rightarrow E_2 = 0$

$$[D] = \begin{bmatrix} 0 & 0 & 0 \\ 0 & 0 & 0 \\ 0 & 0 & \beta G \end{bmatrix}$$



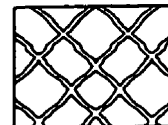
First Crack Closed
Second Crack Open



Both Cracks Closed

- Smeared Cracking Technique - replace G by βG to account for shear stiffness due to aggregate interlock and dowel action

β = shear retention factor, $0.1 \leq \beta \leq 1.0$ (Lin, 1973)



Both Cracks Open

Figure 8.4 Material constitutive model for concrete (NONLACS program)

8.2.4 Application -Finite Element Idealization

Each connection specimen was modelled using quadrilateral shell elements with cubic displacement fields in both directions. Figure 8.5 represents the finite element idealization used for the computer model, with 254 shell elements and 6 boundary elements (to calculate the reactions). These reactions were used to post-tension the W-shape steel beam at specimen boundary to the strong floor of the laboratory. The 193 nodes were numbered in an efficient way to get an optimum stiffness matrix with a minimum band width.

Each node had three degrees of freedom - normal rotation, ω , horizontal displacement, u , and vertical translation, v . A fine mesh was used to model the critical area with stress concentration close to the connection, while a coarse mesh was used in the zone farther away from the connection (Figure 8.5). Trapezoidal transition elements were used to connect the fine elements to the coarse elements. The aspect ratio for the concrete panel elements was maintained near one, except for the concrete element adjacent to the side boundaries.

The thickness of the concrete panel was divided into two 25 mm thick layers. The steel beams (W-shape) at the specimen boundary were modelled by steel plate elements divided into two layers. The thicknesses of these elements were determined based on the moment of inertia and the effective shear area of the steel beams. The two-dimensional steel anchors were idealized using the embedded reinforcement concept, while the steel wire mesh was idealized as a smeared steel layer.

To determine the reactions in the boundary elements, the spring concept (for each support; one horizontal and one vertical spring with high stiffness) was used. The steel face-plate and the angle were modelled using steel plate elements.

Loading was applied in 30 load steps, and the in-plane forces were applied at three nodes, Node 135, Node 150, and Node 159 in the ratio of 50% of the total load for node 150, and 25% of the load at each of the nodes 135 and 159 (Figure 8.5). The total weight of the specimen was imposed in the first load step. Equilibrium was achieved using 15 iterations in each load step. The load steps for the applied forces had large increments at the start and much smaller increments towards the end, to evaluate the

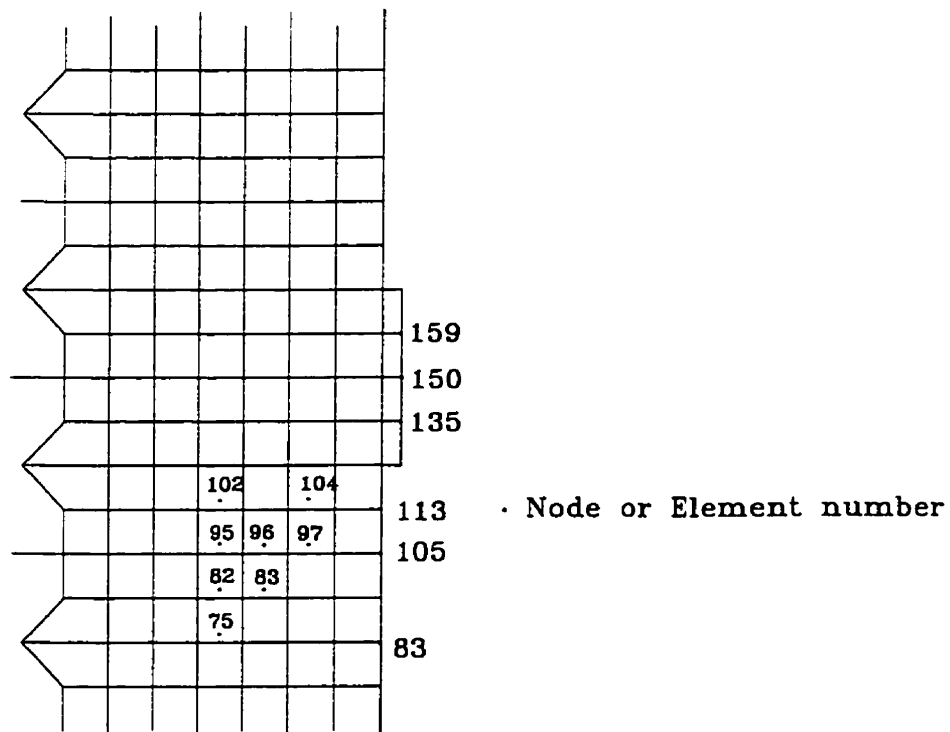
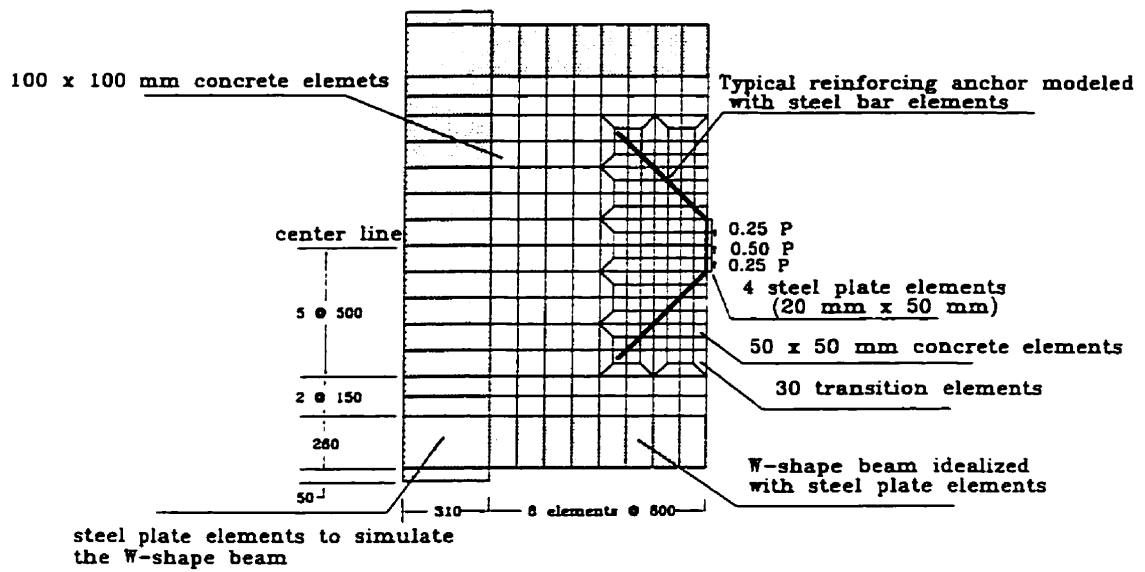


Figure 8.5 Finite element idealization for specimens

behaviour of the structure by taking into account the material non-linearity effects near the ultimate load step. The applied load was increased monotonically and shrinkage, creep and temperature effects were not considered.

The 3 by 3 Gauss integration points for analysis, and the result of the 5th integration point (middle, average point) were used to evaluate the stress and strain in the concrete (cracking pattern) and the overall deflection at Node 150.

8.3 Comparison of Analytical and Experimental Results

8.3.1 Introduction

As mentioned in Section 8.2, the NONLACS finite element analysis program was used to study the non-linear responses of Connections Type A through E, subjected to monotonically increasing loads until failure. The output from the NONLACS analysis presented in this chapter is referred to as "analytical" results. As noted in Chapters 3, 4 and 5, the experimental specimens were subjected to both the monotonically increasing loads and reversed cyclic loads to study their complete responses. In this chapter the experimental results of the connections subjected to monotonically increasing loads are termed "monotonic experimental" results. Strictly speaking, the cyclic experimental results cannot be compared with the "analytical" predictions because the loading imposed on the analytical and cyclic experimental specimens are different- being monotonically increasing loads for the analytical study and reversed cyclic loads for the cyclic experimental investigation. Therefore, the comparison is focussed between the "analytical" and "monotonic experimental" results. The following evaluations are based on analytical predictions (NONLACS results), conventional methods of design, the conceptual models developed herein, and the experimental data. Table 8.1 compares the CPCI, modified CPCI, the method proposed in this research and NONLACS results with the monotonic experimental strengths.

The analytical load-deflection characteristics, the ultimate strength of a connection and the crack patterns provide key information necessary to understand the connection

response. In predicting the load-deflection response of the structure, complete stress-strain relationship for the steel including the strain hardening and non-linear stress - strain relationship for the concrete including the tension stiffening effects should be considered. The constitutive material models used in the NONLACS program were presented in Section 8.2.2. The "analytical" and "experimental" results for the load-deflection characteristics, ultimate strength, response of reinforcing steel anchors, and crack patterns are compared in this chapter. For each element, the stress and the strain at the 5th (middle point) Gauss integration point were selected to evaluate the crack propagation, while the displacement of Node 150 coincident with the location where displacement of test specimens was measured (Figure 8.5), and the strains at Location 1 in the tension zone (Figures 3.2, 3.37, 4.2, 4.33 and 5.2) for the steel anchor were also compared with the experimental results.

8.3.2 Connection Type A

The analytical load-deflection curve for Node 150 is compared with the monotonic experimental displacement response for Specimen AMS in Figure 8.6. The analytical initial stiffness, and the load at which the first cracks initiated, correlate well with the test results. This is basically due to elastic responses of the analytical and the experimental specimens. As can be seen from Figure 8.6, the slope of the monotonic experimental load-deflection curve decreases more after the cracking compared with that for the "analytical" slope. This is basically due to slip of anchor bar and deterioration of the bond at the steel-concrete interface in the "experimental" model, while in the "analytical" model (NONLACS program) the assumption of "perfect" bond between the concrete and the steel causes that the slip of the bar at the concrete-steel to be ignored. The analytical ultimate strength is 190 kN, which is very close to the monotonic experimental value (187 kN). The analytical ultimate displacement predicted to meet the last convergence criterion (0.0005 of the norm of the displacement vector was used as convergence criterion in the NONLACS program) is 1.85 mm, which is 25% smaller than the monotonic experimental results (2.5 mm). The lower strength and the larger monotonic experimental displacement are due to the fact that the monotonic experimental loading causes degradation of bond

strength and stiffness at the interface of the concrete and the reinforcing bar, which in turn reduces the stiffness and the ultimate capacity of the connection.

Comparison of the result from the NONLACS analysis and the conventional method of design (CPCI 1987) shows that the ultimate strength of the connection calculated using the "truss analogy" (131 kN) gives a conservative value of the connection strength. It is interesting to note that the ultimate strength according to the modified CPCI method is 194.5 kN which is just about 2% larger than the analytical value, showing close agreement between the value obtained using modified CPCI method and the NONLACS program. The value calculated using the method proposed in this research program (Section 8.1.1.1) is 185.7 kN which is in an excellent agreement with NONLACS result and "monotonic experimental" value (Table 8.1). The ultimate strength according to the CPCI method (131 kN) is 29 % smaller than the experimental value (Table 8.1).

Analytical load-steel strain curves for the connection are compared with the monotonic experimental results in Figure 8.7. It is seen that up to cracking, the strain plot corresponds well with the load-strain test results. Beyond extensive cracking ($P=90$ kN), the slope of the experimental load-strain curve decreased more compared with the analytical plot, which is due to the deterioration of the bond strength in the test model. It can be noted that the analytical load at which the tensile steel reinforcing bar element yielded was close to 185 kN, while the reinforcing bar in experimental model yielded after a sudden drop in the applied shear force to the value of 112 kN (Figure 8.6). This difference is due to the fact that for experimental specimen, the bond strength at the concrete-bar interface exhausted resulting in the spalling of the concrete, however in the analytical model due to assumption of perfect bond the sudden drop in the load did not occur.

The cracking pattern, at the commencement of yielding correlated well for the "analytical" and "monotonic experimental" models [Figures 8.8(a) and 3.4(a)]. However, the extent of cracking of the concrete in the analytical model was smaller than that for monotonic experimental models near the ultimate load stage [Figures 8.8(b) and 3.4(b)]. This is due to the assumption of perfect bond between the concrete and the steel for both the reinforcing bar anchor and the reinforcing mesh in the "analytical" specimen resulting

in a stiffer model, while in the "monotonic experiment", due to deterioration of bond strength and slip of the bar at the concrete-steel interface, the density of the cracks increased.

Table 8.1 Comparison of "analytical" and "monotonic experimental" shear strengths

Connection Type	Analytical Strength (kN)				Monotonic experimental strength (kN)
	CPCI Method	Modified CPCI Method	Proposed Method	NONLACS Results	
A	131	194.5	185.7	190	187
B	131	196	234.7	240	242
C	128	148.8	149.8	158.5	152
D	203	-	221.3	232.5	226
E	45.4	-	49.4	55	53

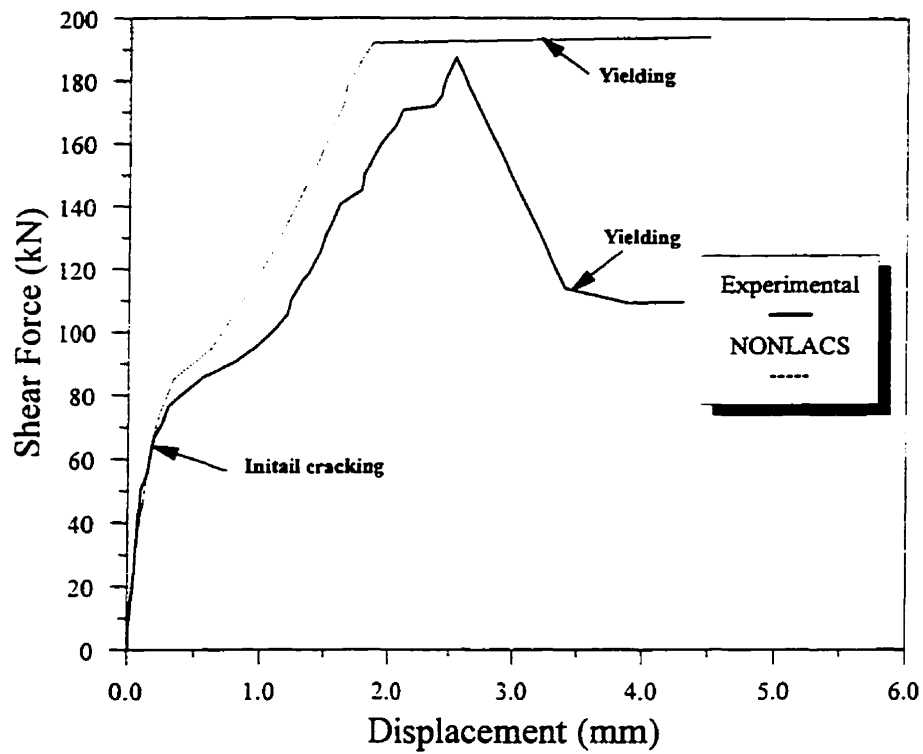


Figure 8.6 Comparison of analytical and monotonic experimental load-deflection response for Connection Type A

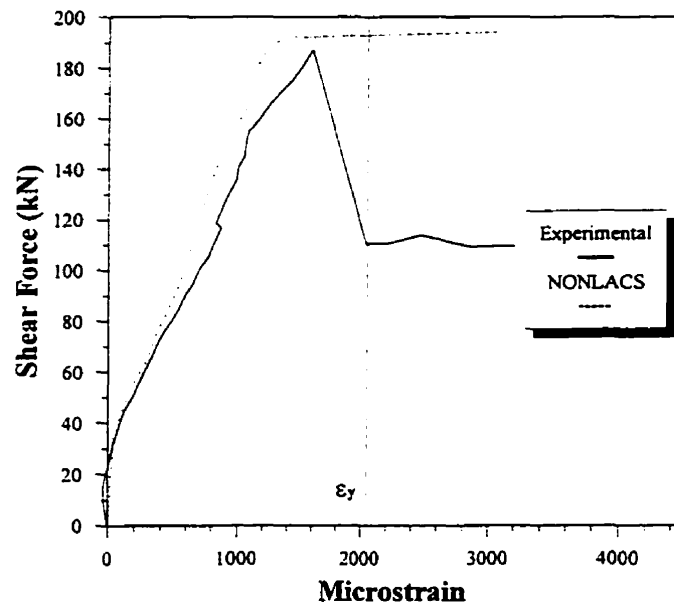
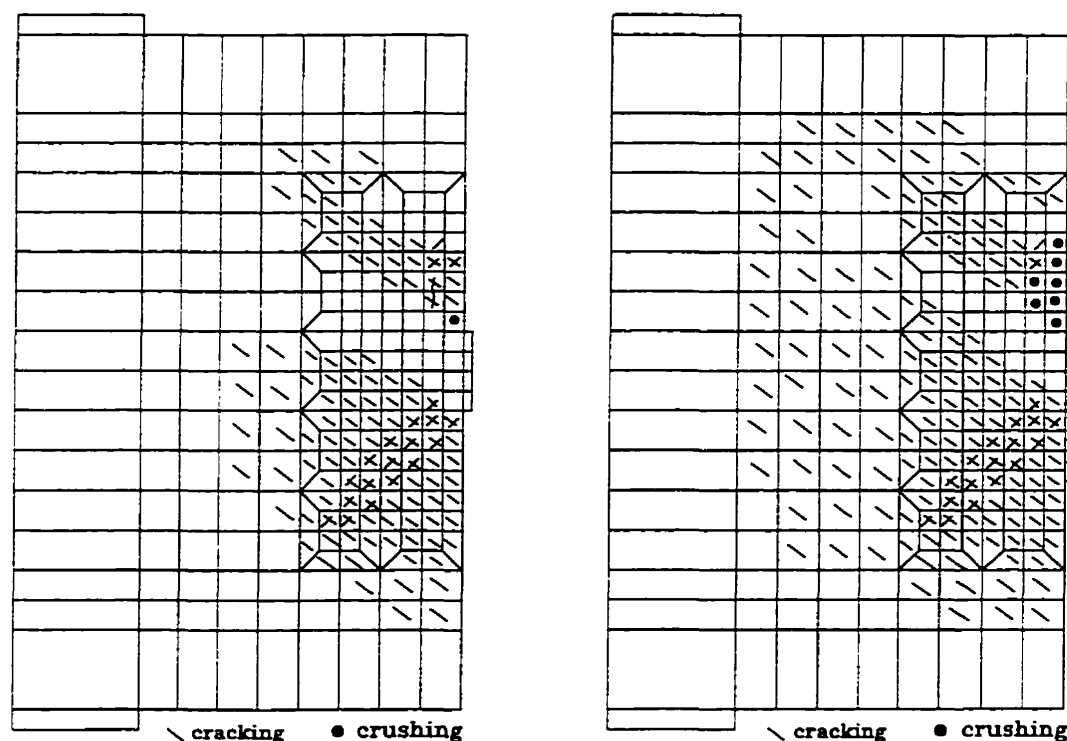


Figure 8.7 Comparison of analytical and monotonic experimental load-strain curve for Connection Type A



(a) Cracking at yield stage ($P=185$ kN)

(b) Cracking at failure ($P=190$ kN)

Figure 8.8 Crack patterns for the Connection Type A

8.3.3 Connection Type B

The monotonic experimental and analytical plots of the applied load P versus the Node 150 displacement are compared in the Figure 8.9. The initial stiffness and the load at initiation of the first cracks correspond well with the monotonic experimental results. This correlation is due to the fact that both specimens were basically in the elastic range before crack initiation. The slope of the load-deflection plot diminishes significantly after the initial cracking, whereas the "analytical" stiffness is not influenced considerably (see Figure 8.9). The stiffness of the "analytical" model decreases significantly at a load value of 130 kN, which was due to a considerable increase in the number of cracked elements in comparison with the previous load step. It was noted that the sharp change of the stiffness in the "analytical" model for Connection Type B appeared at a load greater than that for the Connection Type A. This was due to the 90° hook in the Connection Type B,

which caused the higher strength and stiffness of the Connection Type B compared with those for the Connection Type A. The analytical ultimate load was 240 kN, which is just about 2% lower than the monotonic experimental value. The maximum "analytical" displacement predicted to achieve the specified convergence criterion is 6.1 mm, about 87% of monotonic experimental result. The larger monotonic experimental displacement can be attributed to the slip of the bar anchor at the concrete-steel bar interface, which tends to deteriorate the bond strength and stiffness of the connection.

Comparison of the result from the NONLACS analysis (240 kN) and the ultimate strength calculated according to the modified CPCI method (196 kN) gives a conservative value of the connection strength. The value calculated using the method proposed in this investigation (Section 8.1.1.2 and Appendix A) is 234.7 kN (see Table 8.1), indicating excellent agreement with the NONLACS results (240 kN) and the experimental value (242 kN). The ultimate strength according to the CPCI method of design is 131 kN which is about 45% smaller than the analytical predicted value and the experimental strength (Table 8.1)

The analytical and monotonic experimental results are compared for the applied load versus the strain at Location 1 of the reinforcing steel in Figure 8.10. It can be seen that the analytical strain curve correlates well with the monotonic experimental load-strain curve up to cracking. After cracking, the slope of the analytical load-strain curve was higher compared with that for the experimental curve, and also the analytical load corresponding to yielding of the tensile steel bar was 190 kN, which is about 15% higher than the value obtained experimentally (165 kN). These are attributed to the assumption of the perfect bond in the analytical model which ignores the relative slip at the concrete-steel bar interface.

The "analytical" cracking patterns at the yielding and failure stages are plotted in Figure 8.10. The analytical and monotonic experimental cracking patterns, at the initiation of the first cracks and the yielding stage show good agreement [Figures 3.35(a) and 8.10(a)]. However, the extent of propagation of the cracks in the "analytical" model was smaller than that in the monotonic experimental results, at loads higher than 200 kN and the failure stage [Figures 3.35(b) and 8.10(b)], basically because of the deterioration of

bond at the steel-concrete interface and the resulting deterioration of the specimen under reversed cyclic loading, and the assumption of "perfect" bond between the concrete and the steel in the analytical model

The failure in the "monotonic experimental" and "analytical" models occurred in the tensile zone due to complete deterioration of the concrete along the straight part of the bar and in front of the 90° hook.

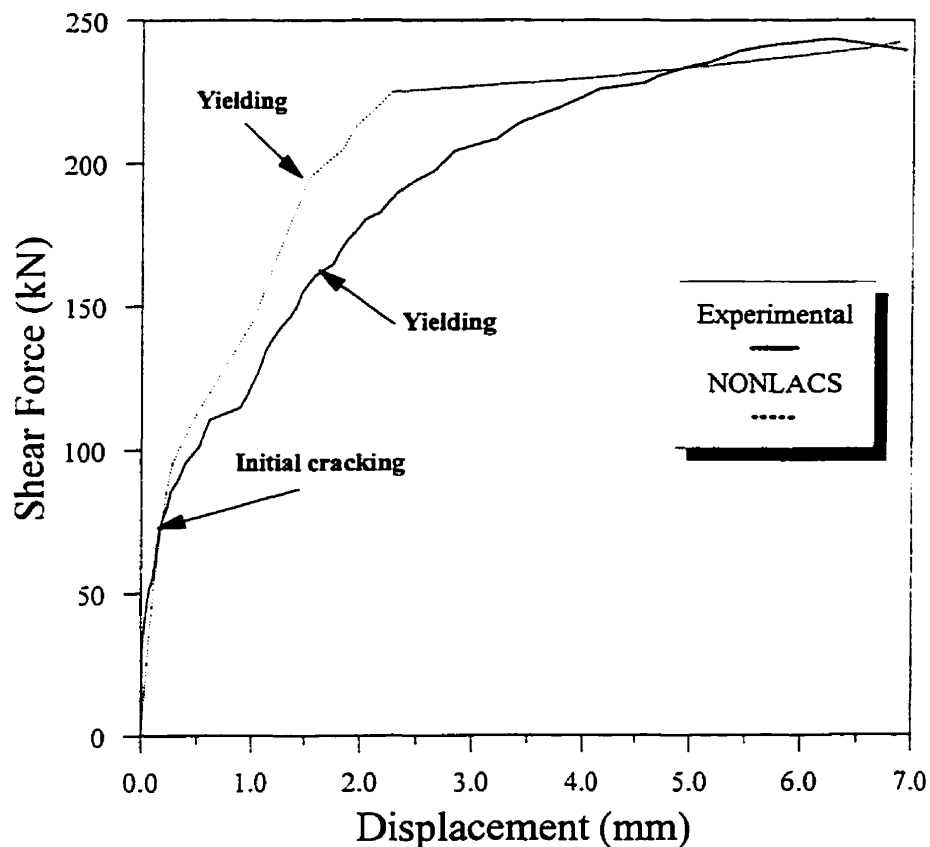


Figure 8.9 Comparison of analytical and monotonic experimental load-deflection response for Connection Type B

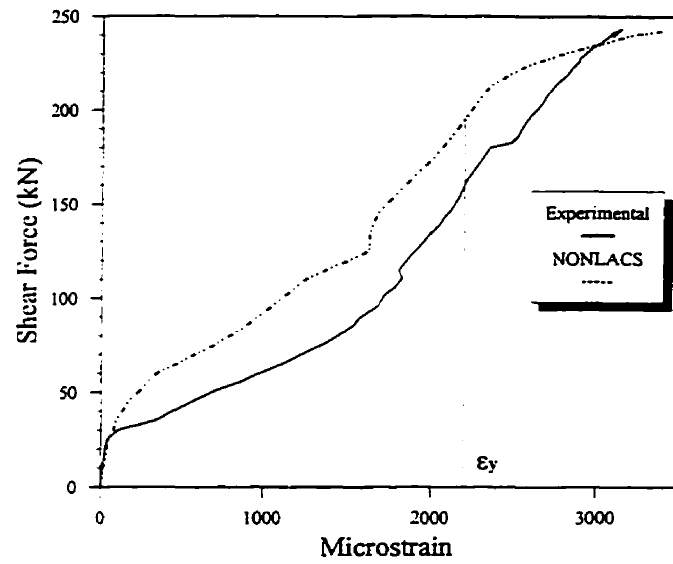


Figure 8.10 Comparison of analytical and monotonic experimental load-strain curve for Connection Type B

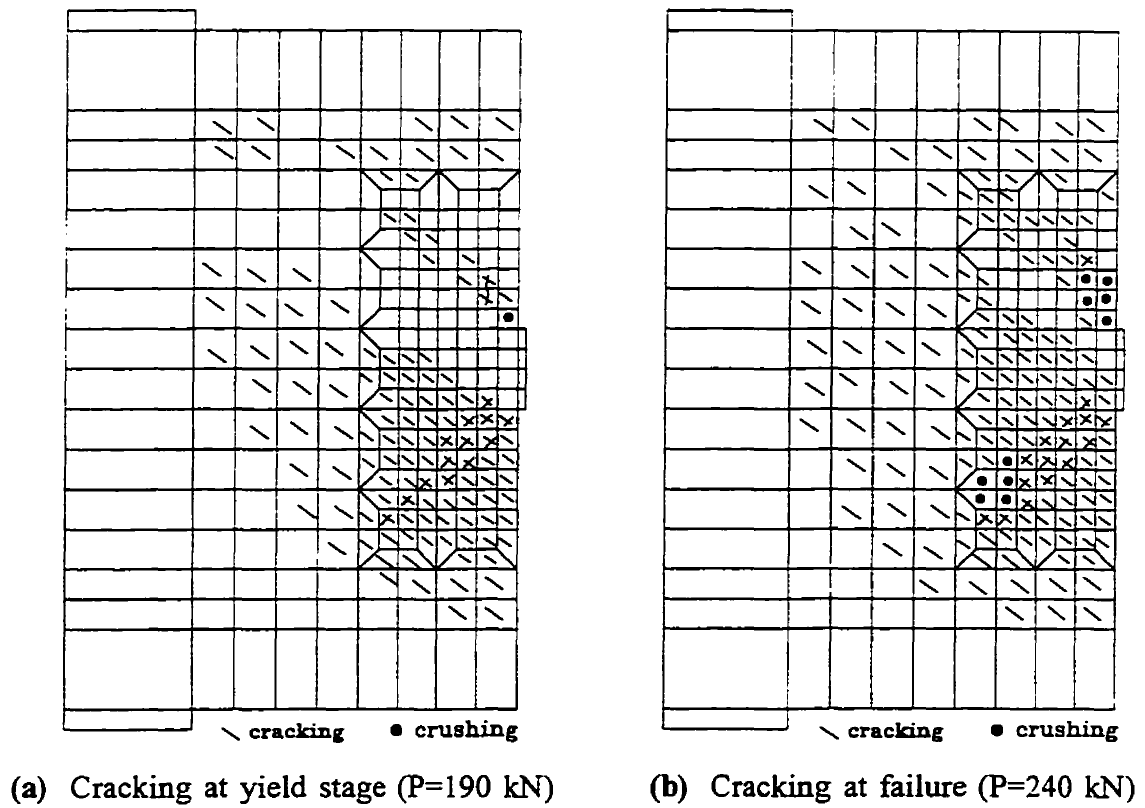


Figure 8.11 Crack patterns for the Connection Type B

8.3.4 Connection Type C

The "analytical" and "monotonic experimental" load-deflection curves at Node 150 for Connection Type C are compared in Figure 8.12. It can be noted that the correlation is generally good. The difference between the "analytical" and the "monotonic experimental" ultimate strength values is about 4%. In addition to the load-deflection curve, Table 8.1 compares the "monotonic experimental" and "analytical" strength values.

The slope of the monotonic experimental load-deflection curve decreases significantly at loads higher than 95 kN, whereas the slope of the analytical load-deflection curve is not significantly affected. The maximum analytical displacement at the last convergence is 2.9 mm which is 70% smaller than the monotonic experimental value. Here again, there are differences between the two sets of results, basically due to the influence of the bond deterioration in the experimental specimen and the assumption of perfect bond between the concrete and the reinforcing steel in the analytical model, resulting in a greater degradation of the strength and stiffness in the "monotonic experimental" model compared with the response of the analytical model.

Comparison of the results from the NONLACS analysis and the conventional design method show that the ultimate strength calculated based on the modified CPCI method and the value calculated using the method proposed in this research program (Section 8.1.1.3 and Appendix A) are 148.8 kN and 149.8 kN, respectively, which agree well with the NONLACS results (158.5 kN, Table 8.1). These results also indicate excellent agreement with the modified CPCI method. The ultimate strength according to the CPCI method of design is 128 kN which is about 19% and 16% smaller than the analytical predicted value and the experimental strength, respectively (Table 8.1)

Figure 8.13 compares the "analytical" and "monotonic experimental" load-steel strain results. The overall analytical response is stiffer than the monotonic experimental response. It can be seen that the reinforcing bar in the "analytical" model yielded at a load value very close to the ultimate load ($P=156.5$ kN), which is about 1.25 times the yielding load in the "monotonic experimental" specimen.

The initiation and propagation of cracks, during the early load steps (up to 60 kN) correspond well for the "analytical" and "monotonic experimental" models. However, for

loads higher than 100 kN, the extent of the "monotonic experimental" crack distribution was larger than the corresponding "analytical" results. The cracking pattern at the yielding and failure stages are presented in Figures 8.14(a) and 8.14(b), respectively. Comparison of the cracking at the yielding stage shows that the density of the cracking in the analytical model was slightly more than that for the experimental due to higher yielding load in the NONLACS results [see Figure 8.14(a) and 4.4 (a)].

Both "analytical" and "monotonic experimental" results show that the failure was governed by the deterioration of the concrete surrounding the anchor bar in the tensile zone and the concrete around the steel angle.

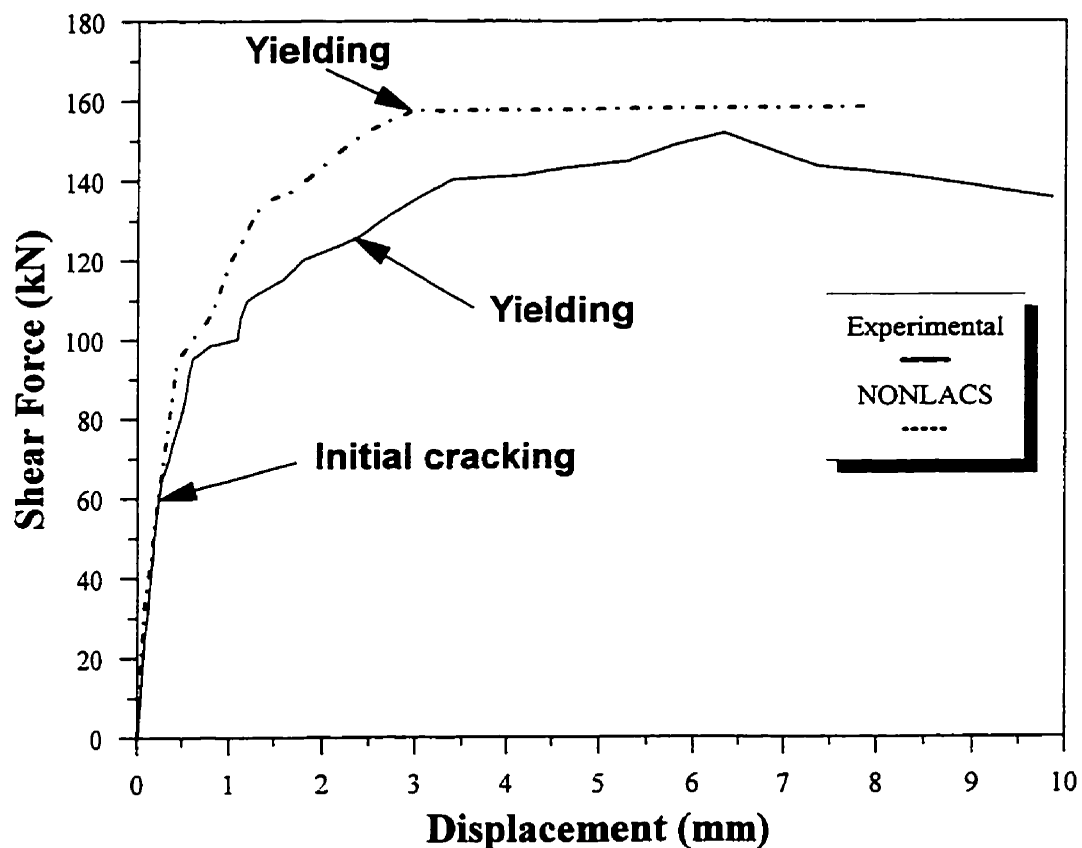


Figure 8.12 Comparison of analytical and monotonic experimental load-deflection response for Connection Type C

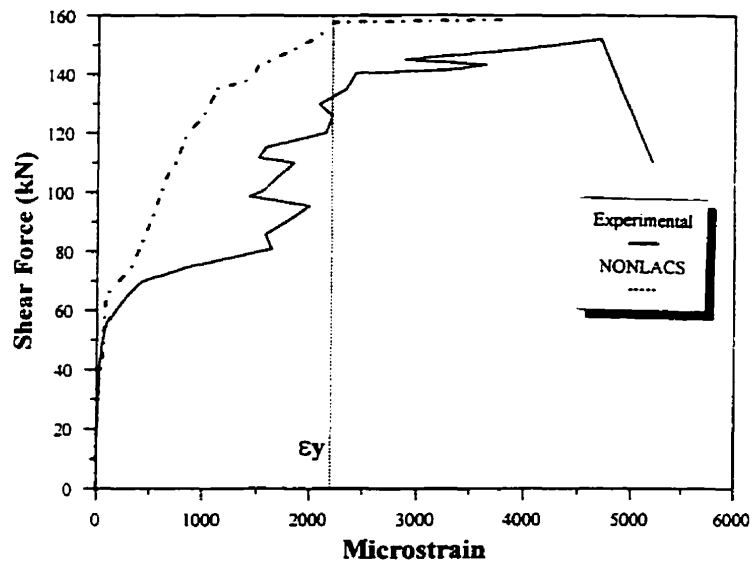


Figure 8.13 Comparison of analytical and monotonic experimental load-strain curve for Connection Type C

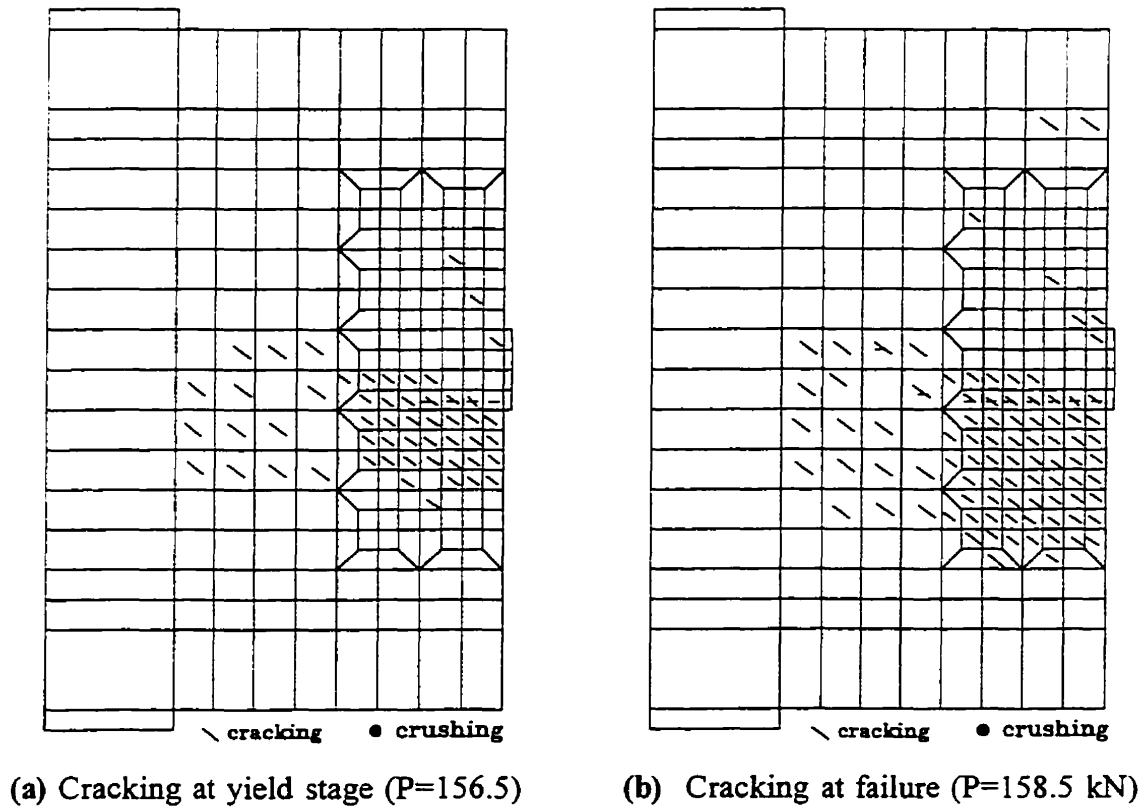


Figure 8.14 Crack patterns for the Connection Type C

8.3.5 Connection Type D

The "analytical" and "monotonic experimental" load-deflection curves for Node 150 are compared in Figure 8.15. The analytical initial stiffness corresponds well with the monotonic experimental results, which is basically due to the elastic behaviour of both specimens up to the initial cracking load. The initial cracking for "analytical" (75 kN) was higher than that for the "monotonic experimental" (62 kN), and as the applied load was increased the "monotonic experimental" stiffness (slope of the load-displacement plot) decreased significantly, while the "analytical" stiffness did not change appreciably. This was due to assumption of perfect bond at the concrete-steel bar interface in the analytical model, which ignores the bar slip which occurred in the experimental model. The load-deflection curves showed that the "analytical" ultimate strength (232.5 kN) is just 3% higher than the "monotonic experimental" value (226 kN). The "analytical" displacement corresponding to the last convergence is 5.2 mm, about 19% smaller than the monotonic experimental value prior to failure (6.4 mm). The overall analytical load-deflection response is stiffer than the monotonic experimental response. These differences are mostly due to the slip of the reinforcing bar at the concrete-steel bar interface in the experimental specimen, resulting in the deterioration of bond strength and stiffness at the interface of steel bar and concrete and significant concrete diagonal cracking at the end of the anchor bars. These effects reduce the stiffness and the ultimate strength of the connection. However, the assumption of the perfect bond in the analytical model results in the prevention of the anchor slip at the concrete-steel interface, and it causes a stiffer load-deflection response.

There are no specific conventional design methods to calculate the strength of the Connection Type D. However, the ultimate strength of the connection was calculated using the shear-friction provisions in the CSA Standard CAN-A23.3-M84 and the CPCI metric design manual (Appendix A), giving the connection shear strength of 203 kN. When compared with the NONLACS result, the calculated ultimate strength is about 12% smaller than the NONLACS value (Table 8.1). Comparison of the NONLACS results and the design method proposed herein (Section 8.1.1.4 and Appendix A) show that the strength calculated using this design method (221.3 kN, Appendix A) is just about 4%

smaller than the corresponding values obtained from the NONLACS program (Table 8.1). The ultimate strength calculated using the method developed is in excellent agreement with the experimental value (226 kN, Table 8.1).

Figure 8.16 compares the "analytical" and "monotonic experimental" load-steel strain curves for Connection Type D. As can be seen, the monotonic experimental load-strain curve is more flexible (softer) than the analytical response. It should be noted that the "analytical" load at which the tensile steel bar yielded was approximately 220 kN, about 1.07 times the "monotonic experimental" value (205 kN). As mentioned earlier, these differences, are mostly due to the steel bar slip during testing the "monotonic experimental" specimen and the assumption of perfect bond between the concrete and the reinforcing steel in the "analytical" model.

The cracking patterns, at the commencement of cracking for the "monotonic experimental" and "analytical" models are in good agreement. However, the extent and propagation of the cracking (at loads higher than 95 kN) in the experimental specimen were larger than those for the analytical model. The density of the cracks at yielding for the "analytical" model was less than that for the "monotonic experimental" [Figures 8.17 (a) and 4.35(a)]. However, the cracking pattern plotted at failure for the "analytical" and "monotonic experimental" models are in very good agreement [Figure 8.17 (b) and 4.35(b)]. The mechanism of failure obtained from the NONLACS program results for the "analytical" model was similar to that for the "monotonic experimental" model. The mode of failure in the "analytical" model was due to extensive deterioration of the concrete along the entire length of the tensile reinforcing bar, and accompanied by complete diagonal cracking in the vicinity of the connection and at the end of the anchor bar in the tensile zone.

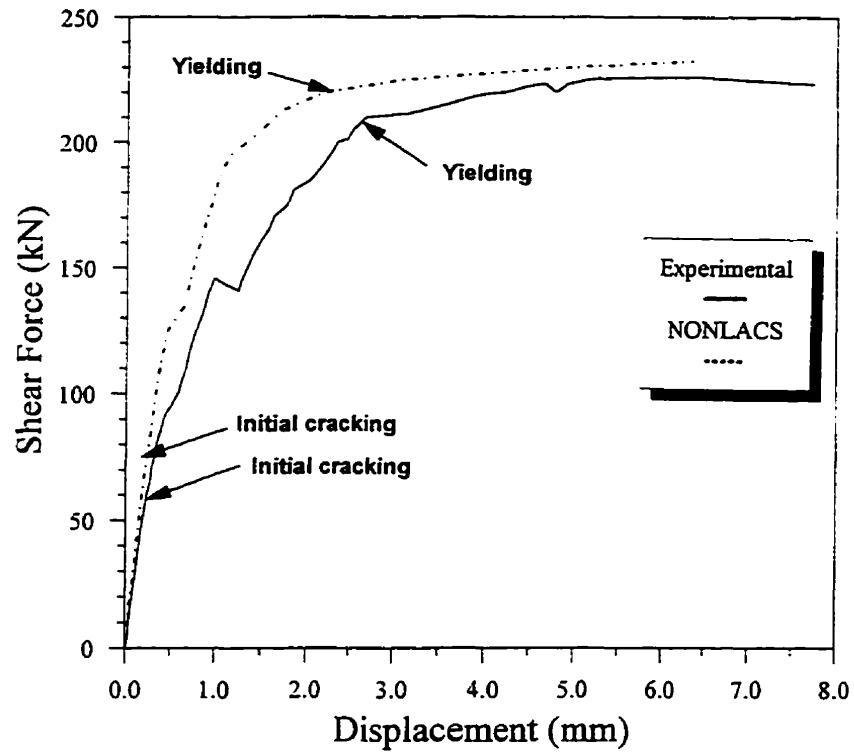


Figure 8.15 Comparison of analytical and monotonic experimental load-deflection response for Connection Type D

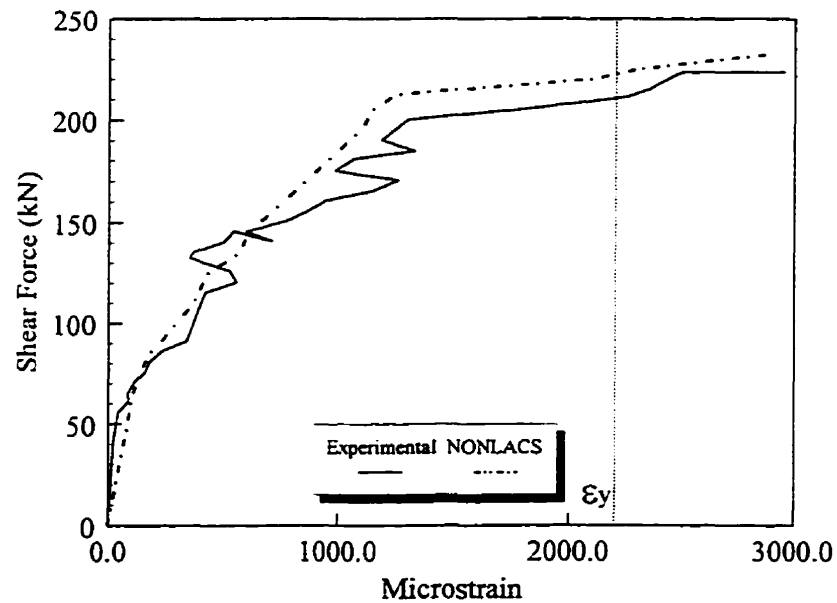
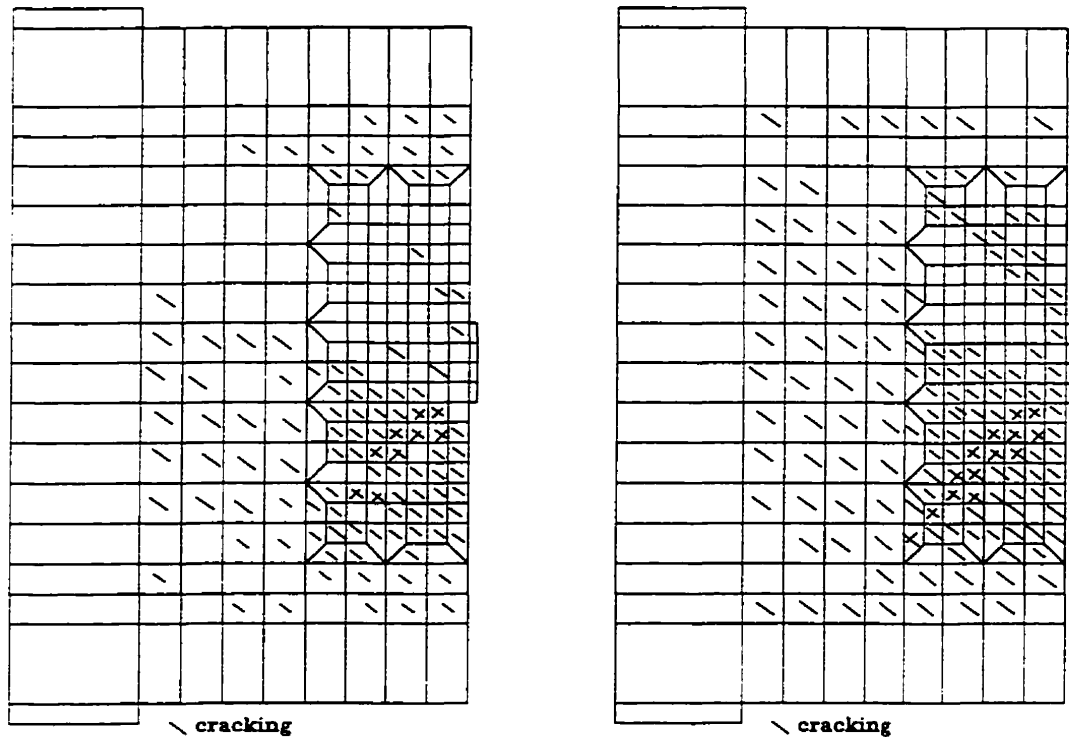


Figure 8.16 Comparison of analytical and monotonic experimental load-strain curve for Connection Type D



(a) Cracking at yield stage ($P=225$ kN)

(b) Cracking at failure ($P= 241$ kN)

Figure 8.17 Crack patterns for the Connection Type D

8.3.6 Connection Type E

The analytical load-deflection curve for Node 150 is compared with the monotonic experimental displacement response for Specimen EMS in Figure 8.18. The analytical initial stiffness, and the load at which the first cracks initiated, correlate very well with the test results. This is basically due to elastic responses of the analytical and the experimental models. As noted in Figure 8.18, the slope of the monotonic experimental load-deflection curve decreases more after the cracking compared with that for the slope of the "analytical" load-deflection curve. This is basically due to the deterioration of bond at the interface of the steel plate and the concrete resulting in slip of the steel plate in the "experimental" model, while in the "analytical" model (NONLACS program) the assumption of "perfect" bond between the concrete and the steel plate causes the slip of the plate at the concrete-steel plate interface to be ignored. The analytical ultimate strength is 55 kN, which is very close to the monotonic experimental value (53 kN). The

analytical ultimate displacement predicted to meet the last convergence criterion is 2.2 mm, which is 30% smaller than the monotonic experimental results (3.1 mm). The slightly lower strength and the larger monotonic experimental displacement are due to: (1) the monotonic experimental loading causing more degradation of the concrete strength and stiffness at the interface of the concrete and the steel plate, and (2) the assumption of bilinear stress-strain curve for the stud in the analytical model influencing the ultimate strength since the connection behaviour after the yielding of the stud and failure mode was completely dependent on the steel stud response. These, in turn, reduce the stiffness and the ultimate capacity of the connection.

Comparison of the results from the NONLACS analysis (55 kN) and conventional method of design show that the ultimate strength of the connection calculated using the "shear-fraction" suggested in the CPCI Manual (45.4 kN) gives smaller strength values. The ultimate strength according to the CPCI method (45.4 kN) is about 14% smaller than the "monotonic experimental" value (Table 8.1). Comparison of the design method proposed in this thesis (Section 8.1.1.5 and Appendix A) with the analytical value show that the strength calculated using this design method (49.4 kN, Appendix A) is about 10% smaller than the corresponding value obtained from the NONLACS program (Table 8.1). The ultimate strength calculated using the proposed method (49.4 kN) is in excellent agreement with the experimental value (53 kN, Table 8.1).

Comparing the "monotonic experimental" and "analytical" strengths show that the ultimate analytical strength is only 4% higher than the "monotonic experimental" value. The lower strength is mostly due to degradation caused by the bond deterioration and the steel stud response. As was observed experimentally, the behaviour of the connection under the in-plane loading was governed essentially by the response of the steel headed studs, which caused stud fracture failure in the connection. The critical stress state of the steel studs resulting in fracture was mostly due to shear.

The analytical load-steel strain curves for the connection are compared with the monotonic experimental results in Figure 8.19. It is seen that the strain plot up to yielding corresponds well with the load-strain test results. The analytical yielding load at which the tensile steel headed stud yielded was 47 kN, which is only slightly higher

than the yielding load in the experimental model (45 kN, Figures 8.18 and 18.19). This difference is due to the fact that for the experimental specimen, the bond strength at the concrete-steel plate interface was exhausted resulting in the separation of the steel plate from the concrete, with all the applied load being carried by the steel studs. However because of the assumption of perfect bond between the concrete and the steel plate in the analytical model, some of the applied load was resisted through the bond at the concrete-plate interface resulting in a higher yielding load in the analytical model.

The cracking pattern, at yielding correlated well for the "analytical" and the "monotonic experimental" models. However, the extent of cracking and spalling of the concrete at the failure stage in the monotonic experimental model was larger than that for the analytical results [Figures 8.20(b) and 3.5]. This is due to the assumption of perfect bond between the concrete and the steel for both the steel plate and reinforcing mesh in the "analytical" specimen, while in the "monotonic experiment" due to deterioration of the bond strength and separation at the concrete-steel interface, the crack density increased.

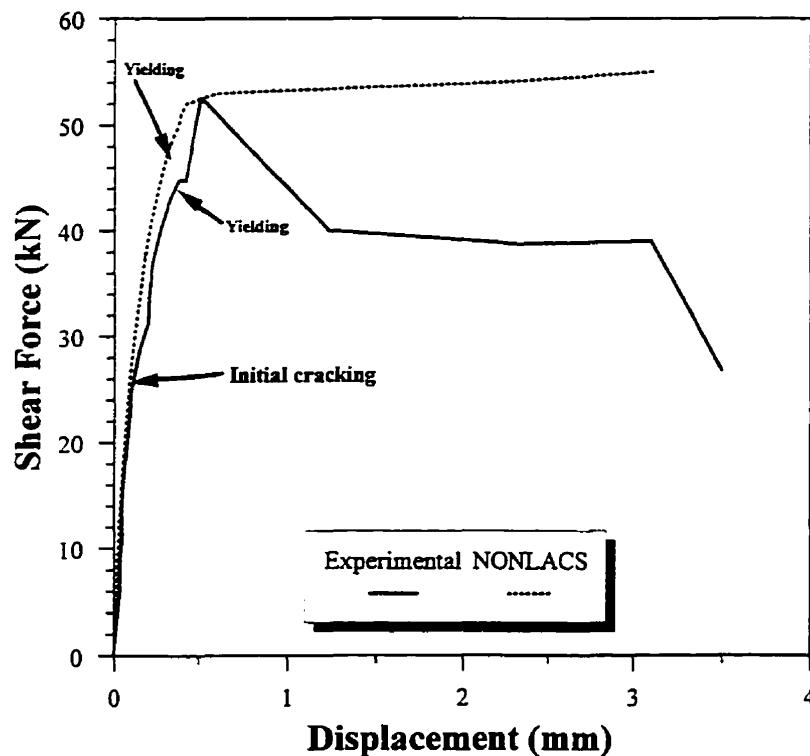


Figure 8.18 Comparison of analytical and monotonic experimental load-deflection response for Connection Type E

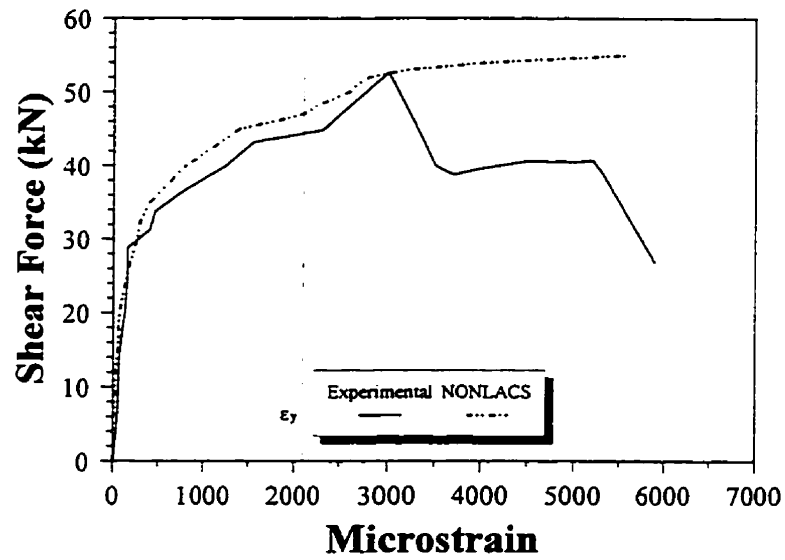
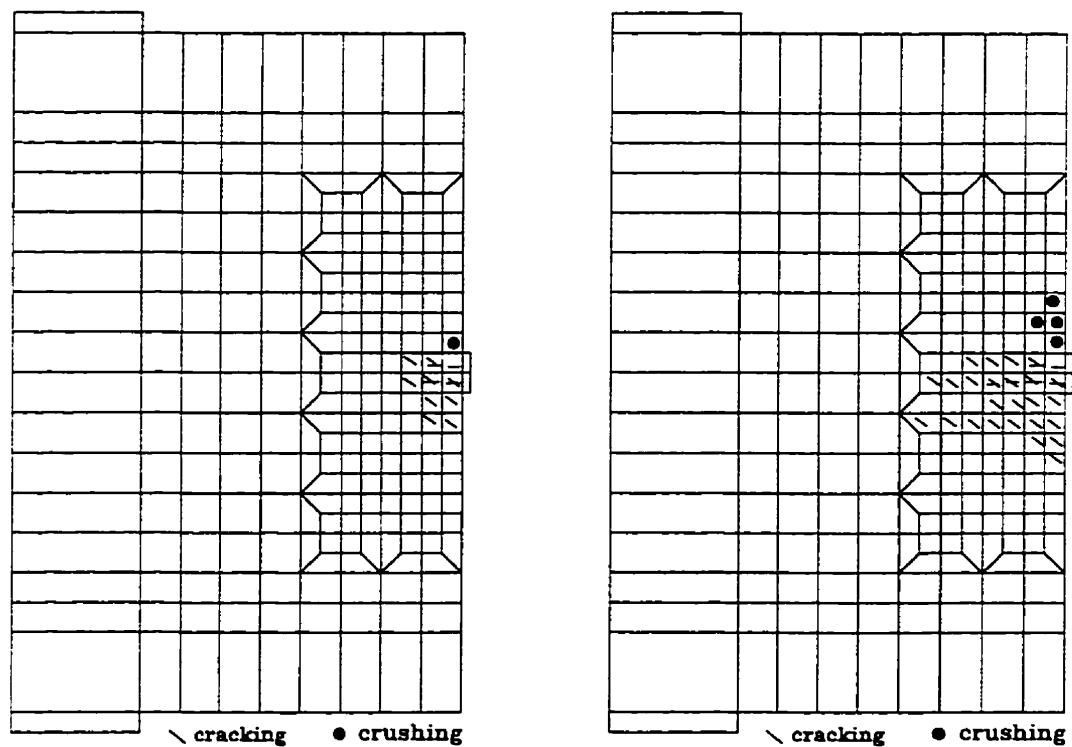


Figure 8.19 Comparison of analytical and monotonic experimental load-strain curve for Connection Type E



(a) Cracking at yield stage (P=47 kN)

(b) Cracking at failure (P=55 kN)

Figure 8.20 Crack patterns for the Connection Type E

8.4 Analytical Nonlinear Response of Connections under Combined Loads

8.4.1 Response Analysis

The nonlinear responses of the specimens for Connection Types A through E have been analyzed for the various combinations of axial and shear loads using the NONLACS program . The resulting axial force - shearing force interaction curves along with the interaction equations are shown in Figures 8.21 to 8.25 for the Connection Types A through E, respectively. The strength interaction analysis results under monotonically increasing loads are summarized in the Table 8.2. It can be noted that the shearing strength decreases when an axial tensile force is monotonically applied , even at small values of the axial tensile force, because the tensile force is transferred to the reinforcing bar by the face plate or the angle causes tensile stresses in the concrete along the reinforcing bar (bond stress) and, also, stresses in the concrete at the interface of the steel angle or the plate, thereby weakening the bond strength when the shear force is applied.

8.4.2 Evaluation of analytical interaction responses with the experiments

There is no rational way to compare the monotonic analytical interaction curve with that for the cyclic test results, and also there are not enough experimental data under combined monotonically increasing loads to be compared with the analytical results. Analysis of the connections under combined cyclic loads are out of scope of this study and can be conducted in future studies. However, the monotonic ultimate shear strengths obtained for the Connection Types A through E using the NONLACS program are in excellent agreement with the experimental results under the monotonic loading, and also the results of computer analysis of different structures subjected to monotonic loading using NONLACS programm have shown the reliability of the NONLACS for accurate prediction of the ultimate strength of the reinforced concrete structures [Ghoneim (1987), Razaqpour et al. (1988 and 1989), Risha (1991), Shariatmadar and Mirza (1992), Bello (1992), Kheyroddin and Mirza (1994, 1995), Kheyroddin (1996), Shayanfar and Mirza (1995), Manatakos (1996)]. Then, the derived analytical curves and the associated equations using the NONLACS program are used as the response of the connection under

the combined monotonic forces to be evaluated versus the cyclic test results. Table 8.2 compares the analytical strength interactions with the experimental cyclic test results.

Analytical (monotonic) and experimental (cyclic) results obtained for Connection Type A are presented in the form of interaction diagrams along with the relevant equations in Figure 8.21. The analytical interaction curve and the equation for the monotonic loading gives higher strengths compared with the measured cyclic test data. Lower test strengths are due to the cyclic loading conditions which deteriorate the connection more compared with the monotonic responses. Table 8.2 indicates that the cyclic strengths of the Connection Type A are about 82 % of the analytical results. Therefore, application of a reduction factor of 0.82 to the analytical equation will give strengths which can be used for calculation of the connection strength under combined in-plane cyclic forces, as shown in Figure 8.21. The modified analytical curve (cyclic) along with the proposed interaction equations for combined cyclic loading conditions for the Connection Type A is shown in Figure 8.21.

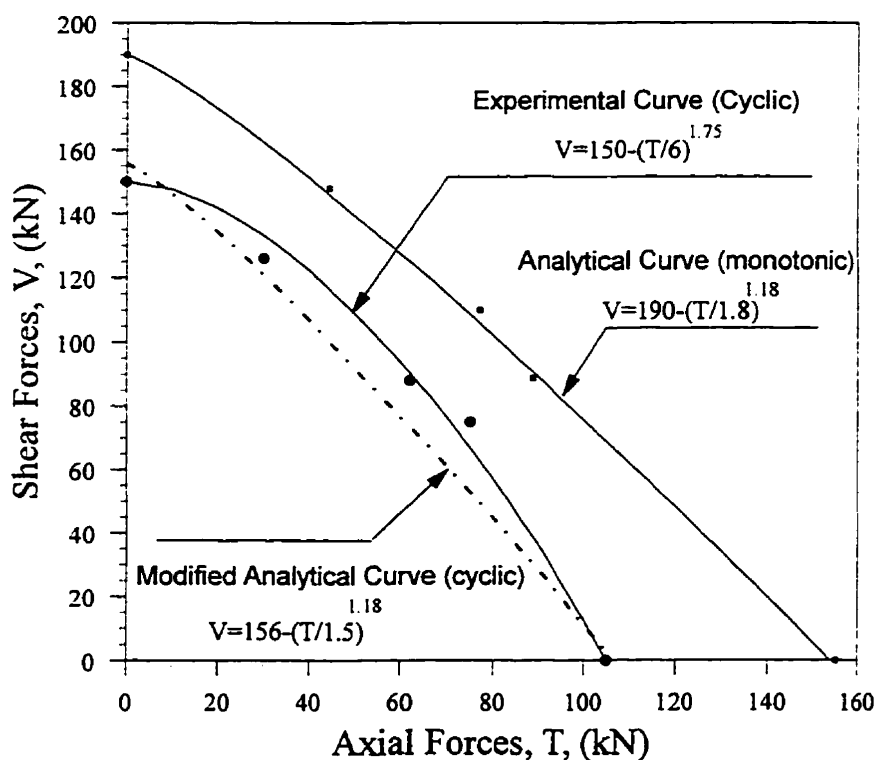


Figure 8.21 Interaction diagrams and equations for Connection Type A

Figure 8.22 shows the analytical and experimental interaction diagrams and the equations for Connection Type B subjected to the monotonically increasing and cyclic loads, respectively. Figure 8.22 shows that for the load combination ratio $T/V=0.7$, the "cyclic" strengths are close to the "analytical" results. However, as expected the "analytical" strength (for monotonically increasing loads) are higher than those for the cyclic test results. Generally, the experimental strengths are about 15% and 8% smaller than the shear and axial analytical results, respectively (see Table 8.2). Therefore, applying a shear and axial reduction factors of 0.85 and 0.9, respectively, in the analytical equation gives a good measure of the strength of the Connection Type B under cyclic loading, (Figure 8.22). The modified analytical curve (cyclic) along with the proposed interaction equations for combined cyclic loading condition for Connection Type B is shown in Figure 8.22.

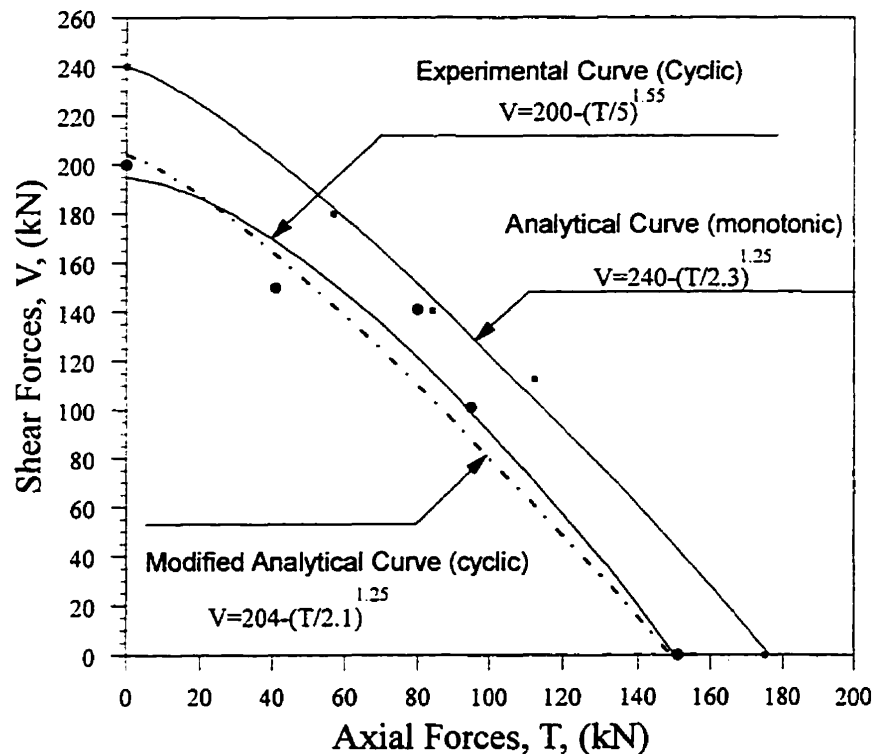


Figure 8.22 Interaction diagrams and equations for Connection Type B

The experimental and the analytical (monotonic) results for Connection Type C are shown in the Figure 8.23 in form of interaction diagrams and the associated equations. The analytical strengths for the combined shear and axial load (T/V) ratios between 0.7 and 1.3 are close to the experimental results. However, the discrepancies are more in the region of the pure shear and axial forces, especially for the pure shear. Table 8.2 shows that in general, the analytical strengths are about 25% and 14% greater than the experimental results for shear and axial strengths, respectively. Therefore, using a reduction factor of 0.85 for shear forces and 0.95 for axial forces in analytical equation for the Connection Type C gives a reliable equation for computing of connection strength under combined cyclic shear and axial loads. This is exhibited as modified analytical curve (cyclic) in Figure 8.23, along with the associated interaction equation for the strength of Connection Type C when subjected to combined in-plane axial and shear loads.

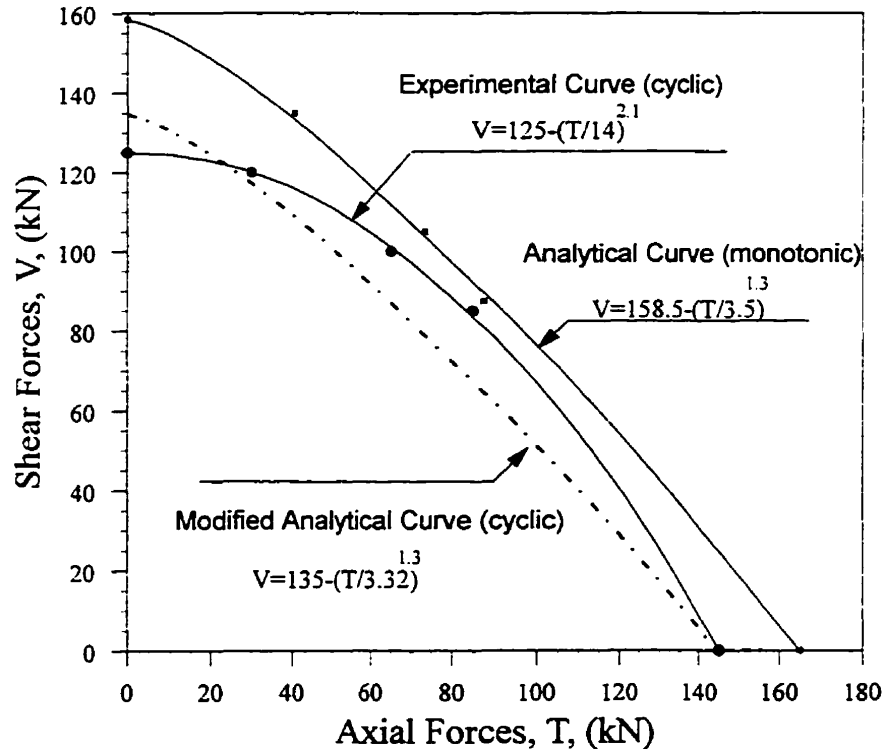


Figure 8.23 Interaction diagrams and equations for Connection Type C

The analytical and cyclic experimental interaction diagrams along with the equations for Connection Type D are shown in Figure 8.24. It is noted that, as expected, the analytical strength obtained from performing a nonlinear finite element analyze under monotonically increasing load gives higher strengths compared with the cyclic test results. Table 8.2 indicates that for interaction results, the experimental strengths are about 0.8 of that for the both shear and axial analytical strengths. Applying a reduction factor of 0.9 to account for the strength degradation due to cyclic loads, the resulting modified interaction curve is in good agreement with the "cyclic" experimental results and give a good assessments for the strength of the Connection Type D under cyclic loading (Figure 8.24). The resulted analytical equation which is appropriate for determination of the connection ultimate strength subjected to combined in-plane seismic loads is also presented in Figure 8.24.

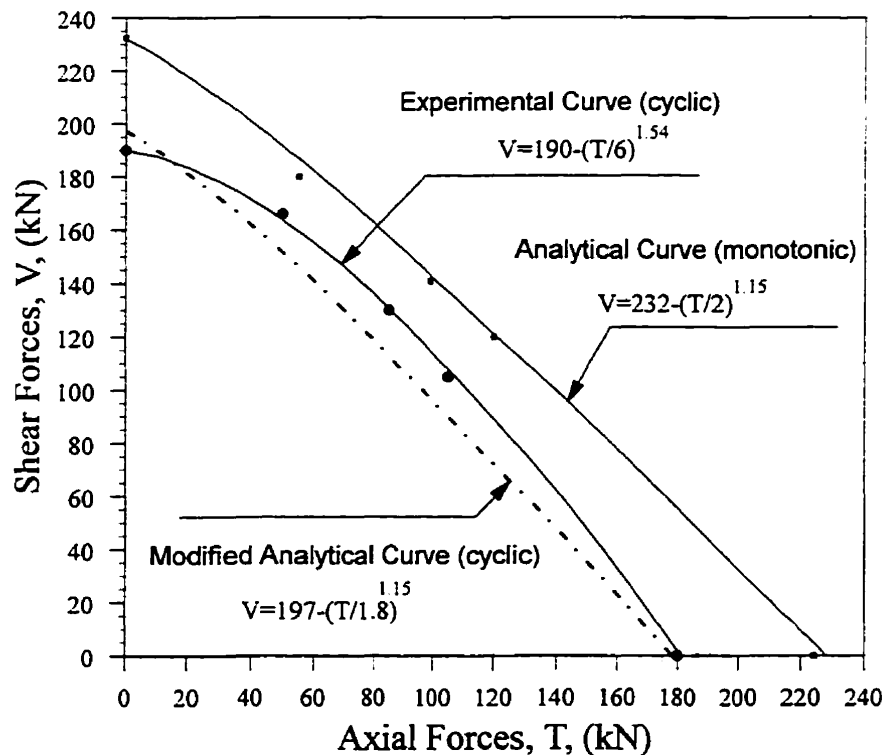


Figure 8.24 Interaction diagrams and equations for Connection Type D

The analytical (monotonic) and experimental (cyclic) results obtained for Connection Type E are illustrated in Figure 8.25. As expected, the "analytical" interaction equation (for monotonically increasing loads) gives higher strengths compared with those for the experimental results for the cyclic loading conditions. It is noted from Table 8.2 that the experimental shear and axial strengths under cyclic loading are about 80% and 70% of the analytical shear and axial results, respectively. It is noted that application of reduction factors of 0.9 and 0.85 to the analytical shear and axial strengths, respectively, provides a good estimate of the strength of the Connection Type E under the combined in-plane cyclic shear and axial forces. The modified analytical curve (cyclic) is shown in Figure 8.25 along with the modified interaction equation for the cyclic loading conditions.

It should be noted that for all types of the connections, for seismic design purposes, the material reduction factor for concrete and steel bar must be applied in the empirical analytical equation for the cyclic loading case which was derived from the monotonic analytical equations using appropriate reduction factors.

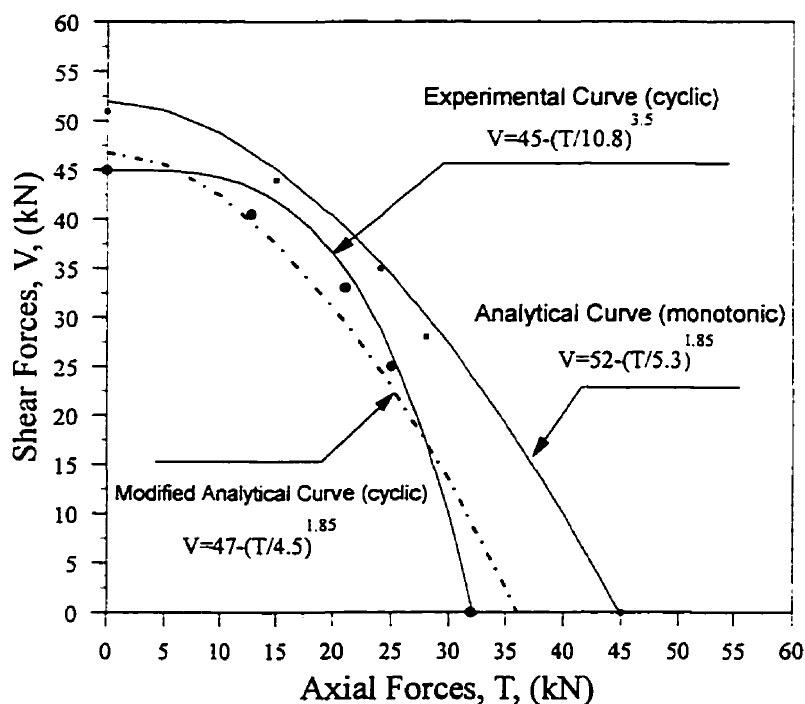


Figure 8.25 Interaction diagrams and equations for Connection Type E

Test Types	T/V Ratio	Ultimate Strengths (kN)			
		Experimental Cyclic Test Results		NONLACS Monotonic Loading Results	
		Shear (kN)	Axial (kN)	Shear (kN)	Axial (kN)
AS	0	150	0	190	0
AST3	0.3	126	30	148	44.5
AST1	0.7	88	62	110	77
AST2	1.0	75	75	88.75	88.75
AT	∞	0	105	0	155
BS	0	200	0	240	0
BST3	0.3	150	41	180	57
BST1	0.7	140	80	140.5	84
BST2	1.0	100	95	112.5	112.5
BT	∞	0	150	0	175
CS	0	125	0	158.5	0
CST3	0.3	120	30	135	40.5
CST1	0.7	100	65	105	84
CST2	1.0	85	85	87.5	87.5
CT	∞	0	145	0	165
DS	0	190	0	232.5	0
DST3	0.3	166	30	180	55
DST1	0.7	130	51	141	99
DST2	1.0	105	63	120	120
DT	∞	0	180	0	224
ES	0	46	0	51	0
EST3	0.3	40	13	44	15
EST1	0.7	35	22	36	24
EST2	1.0	25	25	28	28
ET	∞	0	32	0	45

Table 8.2 Summary of "Analytical" and "Experimental" test results for Connection Types A through E subjected to the combined in-plane forces

Chapter 9

Summary, Conclusions and Design Recommendations

This chapter presents the highlights of the experimental investigation of the proposed connections subjected to monotonically increasing and reversed cyclic loads along with the analytical study of the selected connections undertaken using the NONLACS program.

9.1 Summary

This research program is aimed at studying the complete response and the capabilities of the proposed connections for the precast concrete structures in seismic regions. Although there are empirical methods available for shear design of some of these connections, they are unable to predict the complete response of the connection. The Precast/Prestressed Concrete Institute design method provides a model only for elastic shear design of connection, however, this method cannot predict the inelastic response under monotonically increasing or under reversed cyclic loads, neither for pure shear and pure axial load, nor for combined loading. The embedment length requirement for the reinforcing bar anchor as suggested by the current design method, is based on the bond response of the steel reinforcement under monotonically increasing load when the concrete cover thickness is sufficient (thick panel). However, this research program involved a thin panel (50 mm thickness) for investigation of the complete response of the connections under reversed cyclic loads.

The experimental part of the present work consisted of tests on thirty-one full-scale specimens representing the connections between precast double-tee flanges, or between a double-tee flange and lateral load resisting elements. The boundary conditions for test specimens are determined from the results of analysis of a complete flange of a double-tee. This is achieved by matching the stress conditions in the vicinity of the

connection specimen with the stress conditions in the complete flange of the double-tee. The experimental study is performed with tests of the new proposed connections (anchor bars welded to the steel angle or plate at angles of 45° and 90°, and with 90° standard hooks at the ends, headed studs welded to a steel plate and appropriate combinations of the above anchorage systems) subjected to: (a) monotonically increasing shear forces, (b) reversed cyclic shearing forces, (c) reversed cyclic axial forces, and (d) combination of reversed cyclic shearing and axial forces- all increased gradually in stages until failure using displacement control. The experimental data consisted of load-deflection, load-strain characteristics of steel bar anchors, crack width and crack propagation, energy dissipation characteristics, and mode of failure of the specimens. Furthermore, the experimental results are used to develop anchorage reinforcement patterns aimed at augmenting the strength, stiffness, ductility and the energy dissipation characteristics of the connections.

The effect of reversed cyclic axial force on the different behavioral aspects of the connection, especially the shear response, including the hysteretic responses, load -strain characteristics, crack pattern, cracking, yielding and ultimate loads, mode of failure, energy dissipation and displacement ductility ratios are investigated.

The response comparisons are arranged in three classifications:

- 1) comparison of the response of different connection types (Connection A to E)
- 2) comparison between identical specimens subjected to different loading types (monotonically increasing and reversed cyclic loading).
- 3) comparison between identical specimens subjected to a variety of combined cyclic shear and axial loads (different T/V ratios).

Based on the experimental results, the shearing force - axial force interaction diagrams and the associated equations and their normalized forms are derived for all of the connection types under combined cyclic loads. The design interaction curve and the associated equation for each connection type is developed along with its simplified version and equation for practical design purposes. This provides a practising engineer a useful tool for design of the connections subjected to combined seismic in-plane shearing and axial forces induced in the joints during an earthquake.

On the basis of the resisting mechanism observed experimentally and the physical models describing the ultimate behaviour of the connections, equations are developed for shear and axial strength of the connections under monotonically increasing actions.

Non-linear finite element analyses (NONLACS program) are conducted to study the nonlinear response of the connections subjected to pure monotonic pull-out and shear loads, and combined monotonically increasing loads until failure. The connection steel anchors are idealized using bar elements with elasto-plastic strain hardening characteristics. The nonlinear analysis is performed using an incremental-iterative tangent stiffness approach. The validity of the proposed methods in idealization of connection anchor with the bar element and the applicability and reliability of the NONLACS program for analyzing of the connections are verified by comparison with the experimental results under monotonically increasing shearing forces. The analytical interaction curves and the associated equations under combined monotonic pull-out (axial force) and shear forces are derived. The analytical interaction equations for monotonic loading give higher strengths compared with the measured strength under reversed cyclic loads. The reduction factors to account for the lower strength under cyclic loads are proposed and applied to the analytical equations. The modified analytical curves (for response under reversed cyclic loads) and their associated equations are developed for combined cyclic loading.

In the final stage of this experimental-analytical program, practice-oriented methods are developed to determine the connection strength under shear, axial and combination of these forces at a joint between the elements in a precast system. The simple procedures for design of the connections are presented in form of the equations. Design and detailing guidance are provided for improved connection performance.

9.2 Conclusions and Design Recommendations

The conclusions and design recommendations are based on the experimental results, analytical response predictions and the results of the non-linear analyses of the response of these specimens when subjected to combined monotonically increasing loads.

Table 6.1 summarizes the design and detailing requirements for this new connection types. The conclusions and design recommendations are summarized as following:

9.2.1 Monotonically Increasing Shear Loading

1. Before initial cracking all of the connections displayed high stiffness, which was the highest for the Connection Type B (Specimen BMS) compared with the other connections due to the provision of the hook. The load-deflection curves were linear (Figure 6.1) up to cracking. The cracking load for Specimen BMS was the largest, while the Specimen EMS reinforced with headed studs showed the lowest cracking load (Table 6.1).
2.
 - a) Specimen DMS attained the highest yield capacity followed by Specimen BMS, which was due to the provision of the extra steel bar anchor oriented at 90 degrees.
 - b) Specimen BMS exhibited the largest ultimate strength and displacement ductility ratio followed by Specimen DMS, except that the ductility ratio for Specimen EMS was the largest (Table 6.1). This was due to use of 90° standard hook (as provided in the Connection Type B) and an extra orthogonal steel bar (as utilized in the Connection Type D).
3. The mode of failure for all connections subjected to monotonic loading was due to failure of the concrete, except for the Specimen EMS, which was due to exhaustion of the shear strength of the studs (shear fracture failure), see Table 6.1.
4. The CPCI procedure used to calculate the ultimate strength of the connections subjected to static loading (for the connection types for which it is applicable) gives conservative results.
5. The ultimate strength of the Connection Types A through E can be found using

the methods proposed in this research program as presented in Chapter 8. These methods are developed based on the overall behaviour and failure modes of the connections.

9.2.2 Reversed Cyclic Shear Loading

1. Serious cracking of the anchorage zone was observed before noticeable yielding along the length of reinforcing anchor bar. Any such damage would reduce the likelihood of a good response in a subsequent earthquake.
2.
 - a) Specimen BS exhibited the most cracking, yielding and ultimate strength followed by Specimen DS (Table 6.2). Specimen DS exhibited the best energy absorption characteristics and displacement ductility (Figure 6.3). The use of a 90° hook or/and an extra orthogonal bar at connection centre can improve the overall response of a connection, especially in the Connection Type A.
 - (b) Specimen ES with the headed studs displayed the lowest ultimate strength and the worst energy absorption characteristics compared with all other specimens tested. This specimen failed due to shear fracture of the stud anchors. Therefore, new mechanisms for dissipating energy should be explored. One suggestion is to investigate whether it is possible to design the Connection E such that the energy is dissipated in both the steel headed studs and the surrounding concrete by using studs with larger diameter and length.
3. Comparing the results of the connection subjected to monotonic and cyclic shear loads led to following conclusions:
 - a) The strengths of the connections in the first cycles of loading up to yielding were approximately the same as the strength under monotonically increasing loads.
 - b) The strength and stiffness of the connections decrease beyond the yielding load

with an increase in the number of cycles. The ultimate strength of the connections under cyclic loads reduced to about 80 % to 85% of the values for the monotonic loading. The values of the β ratios, (β = ultimate strength under cyclic loading / monotonic ultimate strength), for all connection types are presented in Table 6.5 along with the comparison of a cracking and the yielding capacities between the monotonic and cyclic loading conditions.

c) To ensure a safe design, it is recommended that the shear strength of a connection under the earthquake loading should be taken as a 80 % of the expected maximum shear capacity of the connection subjected to static loading (Table 6.5).

d) The failure modes for all of the connection types subjected to reversed cyclic shear loads are close to those for monotonically increasing shear loads (static loading). Thus, the expression used to determine the shear strength under static forces can be modified for the state of cyclic loading.

4. The design approach of the CPCI gives a conservative prediction for the ultimate strength of the Connection Types A and B under cyclic shearing loads, while the predicted ultimate strengths of Connection Types C and E are close to the cyclic experimental results. The ultimate shearing strength of the Connection D subjected to cyclic loads is overestimated by the CPCI method.

5) Comparing the response of the Connection Types A (with steel angle) and Ap (with steel plate) subjected to reversed cyclic shear force results in following conclusions:

a) The cracking and yielding loads were smaller for the Connection Ap, however, the ultimate strength of the Connection Ap (with steel plate) was just about 2% larger than that for the Connection A (with steel angle), see Table 6.4.

b) The strength degradation after the post-peak loading was more for Connection Ap (with steel plate), while Connection A (with steel angle) was able to sustain the strength beyond the post-peak load.

c) The energy dissipation was greater for Connection A compared with that for Connection Ap, however, the displacement ductility ratio for the connection Ap ($\mu = 5$) was higher.

d) The connection in which the reinforcing anchor bars are welded to a steel plate (Connection Type Ap) is not recommended, since it resulted in the fracture of the weld at the steel plate-anchor bar interface and caused brittle failure of the connection which is not desirable. The use of a steel angle instead of the steel plate (Connection Types A to D) gives improved response under cyclic loading.

9.2.3 Reversed Cyclic Axial Loading

1. a) Connection Type D subjected to reversed cyclic axial loads (Specimen DT) attained the highest yield and ultimate strengths followed by the Connection Types C and B (Specimens CT and BT, Table 6.3). Connection Type B exhibited the best energy dissipation characteristics and displacement ductility (Figure 6.5).

b) Providing 90° standard hook at the end of the bar anchors (as in Specimen BT) and/or utilizing an additional orthogonal anchor at the connection centre (as in Specimen DT) makes it possible to significantly improve the strength, stiffness, displacement capacity, post-peak load sustainability and energy absorption characteristics.

c) Connection Type E displayed the smallest capability to dissipate energy under cyclic axial load (Specimen ET) which was associated with its smallest ultimate strength, although the displacement ductility ratio was the largest. Utilizing the recommendation provided for the improvement of the performance of Specimen

ES (under reversed cyclic shear loading) makes it possible to improve all aspects of the connection response subjected to cyclic axial loads (Specimen ET).

2. The theoretical strengths of the connections subjected to static pull-out (tensile axial) force using the CPCI methods were larger than the cyclic experimental strengths, except for Specimens BT and ET, for which the experimental strength was higher than the theoretical strengths. The methods proposed in this research program can be used to determine the axial strength of the connection with reasonable accuracy.
3. The failure modes of the Connection types B and D subjected to reversed cyclic axial loads were very close to the failure modes of these connections under reversed cyclic shear loads. Other connection types exhibited different mode of failure for cyclic axial loading and cyclic shear loading.

9.2.4 Combined Reversed Cyclic Loading

Based on the experimental observations and analysis of the test data performed under combined cyclic in-plane shear and axial forces, the following conclusions are drawn:

1. For all connection types subjected to combined reversed cyclic forces with T/V ratios smaller than 0.3, the nature of the hysteretic shear response of the connection was similar to the hysteresis response of that connection subjected to pure reversed cyclic shearing loading. Also, the shear hysteresis curve under combined cyclic loads with T/V ratios higher than 0.3 were completely different from that for the pure cyclic shear loading.
2. a) The cyclic shearing capacity of all connection types decreases when cyclic axial forces are applied simultaneously, however for the Connection Types C and E subjected to combined reversed cyclic forces with T/V ratios less than 0.3, the shear strength of the connection does not decrease significantly.

b) If the cyclic shear loads are applied simultaneously with the cyclic axial loads, the corresponding cyclic axial strength decreases significantly, even with low shear loads

3. The following conclusions are drawn about the performance of each connection type subjected to combined cyclic loading:

a) Connection Type D demonstrates the best capability to dissipate energy, however, in the domain with very high T/V ratios, where the cyclic shear force tends to very small values and cyclic axial force is quite large, the Connection Type B reveals the best cyclic performance (strength, displacement ductility level and energy dissipation).

b) Connection Type C shows very good seismic response in the range of intermediate values of the T/V ratios between 0.3 and 0.7.

c) The dissipated energy was the lowest for Connection Type E, and Connection Type A displays the lowest displacement ductility ratio.

4. The seismic performance for the different specimens in each connection type are examined with respect to the T/V ratio, and the following deductions can be made:

a) In Connection Type A, the energy dissipation and displacement ductility reduced with an increasing T/V ratio.

b) Application of high T/V ratios resulted in lower dissipated energy and ductility in Connection Types B and E, while a large T/V ratio in Connection Type C gives a larger energy dissipation.

c) The energy dissipation for the specimens in Series D (Connection Type D) which exhibited the same mechanism of failure were of the same order.

9.2.5 Interaction Relationships

Safe design of precast structural systems requires accurate evaluation of the strength of the connection under various combination of cyclic in-plane shear and axial forces induced during an earthquake. There are virtually no research data on resistance of the connections in the precast roof and floor systems under combined reversed cyclic loading.

The following conclusions are drawn from the detailed investigation of these test Series:

1. Application of the equation (1-29)

$$\left(\frac{P_u}{P_o}\right)^2 + \left(\frac{V_u}{V_o}\right)^2 \leq 1.0$$

(where, P_u = factored tensile loads, V_u = factored shear loads, P_o = tensile strength of the connection, V_o = shear strength of the connection) suggested for a single stud anchors subjected to combined monotonic pull-out and shear loads cannot be justified from the experimental data generated for the connections under combined reversed cyclic loads. It is even not applicable for the Connection Type E reinforced with the stud anchors. Therefore, a case-by-case determination of the interaction between applied axial and shear loads becomes necessary for design purposes.

2. Connections in precast roof and floor systems are subjected to significant in-plane shear and axial forces during earthquakes, especially in the vicinity of the stiff lateral load resisting elements. From the results of the full-scale combined reversed cyclic loading experiments in this research program, it can be concluded that the cyclic shear capacity of the connection decreases significantly, when cyclic axial forces are applied simultaneously (Table 8.2). The current design method of PCI/CPCI applied to this type of connections does not account for this axial loading. It is necessary to modify the PCI/CPCI equation to account for

this behaviour and develop an expressions for the connections under combined reversed cyclic loading conditions.

3. The dimensional and non-dimensional (normalized) experimental, design and simplified design curves and the equations derived are presented given in Figures 7.2 to 7.6 and in Tables 7.2 and 7.3. These interaction diagrams and the associated equations provide the necessary tools to design the connections subjected to combined reversed cyclic shear and axial forces.
4. The various strength interaction relationships lead to the following conclusions:
 - a) Connection Types D and B show excellent strength under combined reversed cyclic in-plane shear and axial forces. They can be used in areas with severe and moderate earthquakes.
 - b) Connection Types A and C display good ultimate load capacity under seismic forces, and can be utilized in the regions with moderate and low earthquakes.
 - c) Connection Type E with the weakest performance under cyclic loads is suggested to be used in the zones with low seismic risks.
5. Provision of 90° standard hook (as used for Connection B) and providing additional orthogonal bar anchor (as utilized in Connection D) improve all aspects of the connection response when subjected to monotonically increasing loads or combined reversed cyclic loads.

9.2.6 Non-linear Finite Element Analysis, Experimental and Proposed Method

Detailed non-linear analyses were carried out on thirty connections subjected to monotonically increasing loads, consisting of both pure shear or axial (pull-out) and combined shear and axial loads. A comparison of the non-linear analyses and the experimental results led to the following conclusions:

1. The bar and the quadrilateral membrane elements used to idealize the connection steel anchors and concrete panel in the finite element configuration, respectively, accurately model the connection behaviour which led to the following conclusions:

a) The NONLACS program provides a powerful tool to adequately predict the complete deformational response of the connections up to failure under monotonically increasing loads. This includes load-deflection and load-strain characteristics, crack patterns and failure mechanism.

b) The ultimate shear strengths predicted using non-linear analysis (NONLACS program) under monotonically increasing loads were in excellent agreement with those for the experimental results (Table 8.1).

c) The strength of the connections, calculated using the proposed method (Appendix A), are in excellent agreement with the analytical strengths predicted by the nonlinear analysis (NONLACS program)

2. The nonlinear modelling of the connection bar anchor and the concrete panel is an efficient means to study the nonlinear behaviour of a broad range of connections under monotonically increasing combined shear and axial forces.

3. The "modified analytical curves" are developed based on the results of nonlinear analyses and the proposed reduction factors to account for the strength reduction due to cyclic loads. These models describe the interaction relationship between the reversed cyclic shear and axial forces, and can be used to design the connections in precast systems subjected to seismic loading conditions.

Table 9.1 Design requirements and detailing criteria for the connections

Connection Types	Design and Detailing Requirements
Connection A	a) Provide reinforcing bar embedment length, $l_d = 155 d_b / \sqrt{f_c} > 30 d_b$ b) Completely weld the straight and inclined parts of bar to the angle c) Use a steel angle of thickness, $t \geq 3/4 d_b$ d) Confine the connection partially by providing a double steel mesh around the connection steel anchors
Connection B	a) Provide reinforcing bar embedment length based on: $l_{hd} \geq 150 d_b / \sqrt{f_c}$ (with hook at the end) b) Completely weld the straight and inclined parts of bar to the angle c) Use a steel angle of thickness, $t \geq 3/4 d_b$ d) Confine the concrete surrounding the standard hook
Connection C	a) The embedment length must be $l_d = 155 d_b / \sqrt{f_c} > 30 d_b$ b) Use steel angle with thickness, $t \geq 3/4 d_b$ c) Provide confining reinforcing bar (smaller size) normal to the connection anchors to counteract the splitting of the concrete
Connection D	a) Provide reinforcing bar embedment length, $l_d = 155 d_b / \sqrt{f_c} > 30 d_b$ b) Use steel angle with thickness, $t \geq 3/4 d_b$
Connection E	a) Detail the joint with longer and larger (diameter) headed studs welded to the plate at larger centre to centre distances which will result in a higher strength b) Use steel plate thickness, $t \geq 4/5 d_b$

9.3 Areas for Future Research

Some areas requiring further investigation are as follows:

- i) Very little experimental data is available on the behaviour of thin flange connections subjected to combined reversed cyclic loading. More experimental work should be undertaken to study the influence of the different parameters such as the reinforcing bar diameter and yield strength, concrete strength and cover thickness, stud size (including length and diameter) with respect to the type of loading (monotonically increasing or reversed cyclic loading involving shear, tension, or combined shear and tensile loads) to improve the understanding of the behaviour of these connections under seismic loadings.
- ii) Methods of retrofitting strength-deficient connections, which address the need to improve partial confinement, limit the concrete spalling and the out of plane rotation of the connection require investigation. This would permit the retrofitting of the connections with inadequate seismic performance in moderate and severe earthquake regions.
- iii) Derive load-deflection relationship models for the connections under combined in-plane reversed cyclic loading. Develop a new type of linkage element based on the derived cyclic models, and implement it in a nonlinear finite element program. The resulting nonlinear program with the cyclic linkage element capability would be suitable for the nonlinear dynamic analysis and design of the precast concrete structures subjected to seismic loads.

Statement of Originality

The original contributions in the research programm reported are:

1. Thirty full-scale specimens of five (existing and new) types of connections in precast roof and floor systems, were constructed and tested under:
 - i) monotonically increasing shear loading
 - ii) reversed cyclic shear loading
 - iii) reversed cyclic axial loading, and
 - iv) various combinations of reversed cyclic shearing and axial loadto determine their complete responses. Also, one full-scale specimen (Connection Ap) was built and subjected to reversed cyclic shear loads in order to investigate the efficiency of utilizing the steel angle versus the steel plate, thereby preventing the undesirable weld fracture. To the best of the author's knowledge, this is the first experimental work on these types of connections subjected to reversed cyclic loading.
2. The influence of the simultaneous application of reversed cyclic shear and axial forces on the different behavioral aspects of the connection performance (deformability, ductility level, dissipated energy and strength) are investigated, especially the effect of the axial load on the shear response, and vice versa.
3. Based on the experimental results, shearing force - axial force interaction curves and the associated equations are derived for the five different types of connections under combined reversed cyclic loadings. Furthermore, design and simplified design models in form of interaction relationships (including the normalized version) are developed along with the associated equations. These provide a practical tool to design the connection subjected to earthquake loadings. No interaction equations or curves are currently provided for the design of these types of connections subjected to monotonically increasing or reversed cyclic axial and shear loadings in the design codes.

4. Based on the performance at the ultimate load, conceptual models in the form of equations are proposed in Chapter 8 for accurate prediction of the ultimate strength of the connections subjected to static loads.
5. A total of thirty nonlinear analyses, utilizing the embedded reinforcement concept (bar element) for the connection anchors, were performed on each of five connection type to assess the performance of the proposed connections under combined monotonic pull-out (axial) and shear loads. Also, "modified analytical curves" accounting for the strength reduction due to reversed cyclic loads are proposed, which provide a method to determine the strength of the connections under simultaneous application of cyclic shear and axial forces.
6. Recommendations are made for design and detailing of these connections for seismic loading. Presently, the design codes provide no guidance on the subject.

References

- ACI Committee 318 (1989) *"Building Code Requirements for Reinforcing Concrete (ACI 318 - 89) and Commentary - ACI 318 R - 89"*. American Concrete Institute, Detroit, 360 p.
- ACI Committee 408, (1989) *"Bond and Development Lengths"*. Subcommittee on Repeated Loading State-of-The-Art Report on Bond Under Cyclic Loading, Final Draft, RTI-July.
- ACI Committee 224, (1992) *"Cracking of Concrete Members in Direct Tension"*. ACI Manual of Concrete Practice, ACI 224.2R-92.
- ACI Committee 408, (1995) *"Bond and Development Lengths: Bond Under Cyclic Loading"*. ACI 408,2R.
- ACI Committee 408, (1991) *"Abstract of: State-of-the Art-Report: Bond under Cyclic Loads"*. ACI Materials Journal, November-December, pp. 669-673.
- ACI Committee 408, (1979) *"Suggested Development, Splice, and Standard Hook Provisions for Deformed Bars in Tension"*. Journal of Concrete International, July, pp. 44-61.
- American Society of Civil Engineers, ASCE, (1982) *"State of -The- Art Report, Finite Element Analysis of Reinforced Concrete"*. New York.
- Associate Committee on the National Building Code, (1990) *"National Building Code of Canada 1990"*. National Research Council of Canada, Ottawa, Canada.
- Aswad, A., (1979) *"Selected Precast Connections: Low - Cycle Behaviour and strength"*. Proceedings of the Third Canadian Conference on Earthquake Engineering, Vol. 2, Montreal, pp. 1201-1224.
- ATC-3 (Applied Technology Council), (1987) *"Tentative Provisions for the Development of Seismic Regulations for Buildings"*, National Bureau of Standards, U.S. Department of Commerce, Washington, DC.
- Balazs, G.L., (1991) *"Fatigue of Bond"*, ACI Materials Journal, November-December, pp. 620-629.
- Becker, James M., and Sheppard, D.A., (1981) *"Connection for seismic resistant precast concrete connection. In Design of Prefabricated Concrete Buildings for Earthquake Loads"*. Applied Technology Council, ATC-8, pp. 657-683.

- Bello, H., (1992) *"Nonlinear Finite Element Analysis of Planar Reinforced Concrete Structures"*. M. Eng. Project, McGill University, Montreal, Canada.
- Berhe, A.T., (1992) *"Seismic Analysis of One-Storey Precast Concrete Building Using the Response Spectrum Method."*, M. Eng. Project Report, McGill University, Montreal, Canada.
- Canadian Prestressed Concrete Institute, CPCI, (1987) *"Metric Design Manual - Precast and Prestressed Concrete."*, Ottawa, Canada, Second Edition.
- Canadian Standard Association, CSA, (1989) *"Steel Structures for Buildings - Limit States Design, CAN3-S16.1-M89"*. Rexdale, Canada.
- Canadian Standard Association, *"Weldable Low Alloy Steel Deformed Bars for Concrete Reinforcement, G30.16 - M"*. Canadian Standards Association, Rexdale, Canada.
- Canadian Standard Association, CSA, (1994) *"Design of Concrete Structure for Buildings. CAN3 - A 23.3-M84."*, Rexdale, Canada.
- Davies G., Elliott K.S., and Omar W., (1990) *"Horizontal Diaphragm Action in Precast Concrete Floors"*. The Structural Engineering Journal, Vol. 68, No. 2, pp. 25-33.
- Elgabry, A.A., and Ghali. A., (1987) *"Tests on Concrete- Slab Connections With Stud_Shear Reinforcement Subjected To Shear-Moment Transfer"*. ACI Structural Journal, September-October, pp. 433-442.
- Englekirk, R. E.S., (1980) *"Controlling the Seismic Behaviour of Precast Concrete"*. PCI Journal, Vol. 25, No. 2, pp. 72-92.
- Ghoneim, G.A.M., (1978) *"Nonlinear Finite Element Analysis of Concrete Structures"*. Ph.D. Thesis, Department of Civil Engineering, University of Calgary, Canada.
- Ghoneim, G.A.M. and Ghali A. (1982) *"Nonlinear Analysis of Concrete Structures"*. Canadian Journal of Civil Engineering, Vol. 9, pp. 489-501
- Harris, H.G., and Lyengar, S., (1980) *"Full-Scale Tests on Horizontal Joints of Large Panel Precast Concrete Buildings"*. PCI Journal, Vol. 25, No. 2, pp. 72-92.
- Hawkins, Neil M., (1977) *"Seismic resistance of prestressed and precast concrete structures. Part 1- Prestressed Concrete"*. Prestressed Concrete Institute Journal, 22(61), pp. 80-110.
- Hawkins, Neil M., (1987) *"Seismic resistance of prestressed and precast concrete structures. Part 2 - Prestressed Concrete"*. Prestressed Concrete Institute Journal, 23(1), pp. 40-58.

- Hawkins, Neil M., and Lin L.J., (1979) *"Bond Characteristics of Reinforcing Bars for Seismic Loadings"*. 3rd Canadian Conference in Earthquake Engineering, Proceeding Vol. 2, Montreal, Canada, pp. 1225-1252.
- Hu, H.T., and Schenobich, W.C. (1990) *"Nonlinear Analysis of Cracked Reinforced Concrete"*. ACI Structural Journal, Vol. 87, pp 199-207.
- Imran, I., and Panatazopoulou, S.J., (1991) *"Experimental Study of Slab-Wall connections"*. Proceedings of 6th Canadian Conference in Earthquake Engineering, Canada, pp. 647-654.
- Jimenez, R., White, R.N., and Gergery P., (1979) *"Bond and Dowel capacities of Reinforced Concrete"*, ACI Journal, Symposium Paper, 76-4, pp. 73-93.
- Johal, L.S., and Hason, N.W., (1982) *"Design for Vertical on Horizontal Connections in Large Panel Structures"*. PCI Journal, Vol. 27, No. 1, pp. 62-78.
- Kallros, M., (1987) *"An Experimental Investigation of Behaviour of Connections in Thin Precast Concrete Panels under Earthquake Loading"*, M.A.Sc. Thesis, University of British Columbia,
- Karsan, I.D., and Jirsa, J.O., (1969) *"Behaviour of Concrete under Compressive Loadings"*, Journal of the Structural Division, ASCE.
- Kheyroddin, A. (1996) *"Nonlinear Finite Element Analysis of Flexure-Dominant Reinforced Concrete Structures"*. Ph.D. Thesis, McGill University, Canada, 290p.
- Kheyroddin, A. and Mirza, M.S. (1995) *"Parametric Studies of Flexural Rigidity of Reinforced Concrete Beams"*. Structural Engineering Report No. 95-2, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, 109p.
- Kheyroddin, A. and Mirza, M.S. (1994) *"Investigation of Flexural Rigidity of R C Beams Using Nonlinear Finite Element Analysis"*. Structural Engineering Report No. 94-1, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, 143 p.
- Klingner, R.E. and Mendonca, J.A. (1982) *"Shear Capacity of Short Anchor Bolts and Welded Studs: A literature Review"*. ACI Structural Journal, 1982, pp. 339-349.
- Klingner, R.E., Mendonca, J.A., and Malik, J.B. (1982) *"Effect of Reinforcing Details on the Shear Resistance of Anchor Bolts Under Reversed Cyclic Loading"*. ACI Structural Journal, pp. 3-12.
- Kunze, W.E., Sbarounis, J.A., and Amerhein, J.E., (1965) *"The March 27 Alaskan Earthquake -Effeson Structures in Anchorage"*. ACI Journal, Vol. 62, No. 6, pp.

- Libby, J.R., (1992) *"Modern Prestressed Concrete - Connections for Precast Members"*. 4th Edition, PP 536-566.
- Libby, J.R., (1992) *"Modern Prestressed Concrete - Roof and Floor Framing Systems"*. 4th Edition, PP 567-582.
- Mahin, S.A., and Lin, J., (1983) *"Construction of Inelastic Response Spectra for Single-Degree-of-Freedom System"*. Report No. UCB/EERC-80/38, Earthquake Engineering Research Centre, College of Engineering, University of California, Berkeley.
- Manatakos, C.K., (1996) *"Behaviour and Design Of Reinforced Concrete Core-Slab-Frame Structures"*. Ph.D. Thesis, McGill University, Canada, 290 p. + illustrations.
- Martin, L.D., and Korkosz, W.J., (1982) *"Connections for Precast Prestressed Concrete Buildings- Including Earthquake Resistance."* Prestressed Concrete Institute, PCI, Chicago, U.S.A., TR - 2 - 82, 297 p.
- Neille, D.S., (1977) *"Behaviour of Headed Stud Connections for Precast Concrete Panels under Monotonic and Cycled Shear Loading"*. Ph.D. Thesis, University of British Columbia.
- Oliva, M., Gavrilovic P., and Clough R.W., (1990) *"Seismic Testing of Large Panel Precast walls: Comparison of Psedostatic and Shaking Table Tests"*. Journal of Earthquake Engineering and Structural Dynamics, Vol. 19, pp. 859-875.
- Pall, A.S., and Marsh, C., (1979) *"Seismic Response of Large-Panel Structures Using Limited-Slip Bolted Joints"*. 3rd Canadian Conference in Earthquake Engineering, Proceeding Vol. 2, Montreal, June, 1979, pp. 899-916.
- Parmar, S.S., (1987) *"2-D Non-Linear Seismic Analysis of One-Storey Eccentric Precast Concrete Building."* M. A. Sc. Thesis, University of British Columbia., 167 p.
- Paulay, T., and Priestley, M.J.N., (1992) *"Seismic Design of Reinforced Concrete and Masonry Buildings"*, John Wiley & Sons, Inc. New York, 744 p.
- Powell, J.B., Burdette, E.G., and Oluokun, F.A., (1992) *"Prediction of Shear Capacity of Channel Anchors in Thin Slabs"*. ACI Structural Journal, pp. 564-568.
- Precast/Prestressed Concrete Institute, (1985) *"PCI Design Hand book - Precast and Prestressed Concrete"*. Prestressed Concrete Institute, Chicago, U.S.A., Second Edition.

- Precast/Prestressed Concrete Institute, (1971) *"PCI Design Handbook - Precast and Prestressed Concrete"*. Prestressed Concrete Institute, Chicago, U. S. A., First Edition.
- Precast/Prestressed Concrete Institute, (1988) *"Design and Typical Details of Connection for Precast and Prestressed Concrete"*. Chicago, U.S.A, Second Edition.
- Rasouli, A.A., (1991) *"Seismic Analysis of One-Storey Precast Concrete Building"*. M. Eng. Project, McGill University, Montreal, Canada, 177 p.
- Razaqpur, A.G., and Nofal M.E., (1988) *"Transverse Load Distribution at Ultimate Limit States in Single Span Slab-on-Girder Bridges With Compact Steel Sections."* Technical Report MISC-88-01, The Research and Development Branch Ministry of Transportation, Ontario, Canada.
- Razaqpur, A.G., Nofal M.E., and Mirza, M.S., (1989) *"Nonlinear Analysis of Prestressed Concrete Box Girder Bridges Under Flexure."*, Canadian Journal Of Civil Engineering, Vol. 16, No. 6, PP. 845-853.
- Risha, W.A., (1991) *"Analysis and Design of Reinforced Concrete Coupled Structural Walls"*. M. Eng. Project, McGill University, Montreal, Canada.
- Saffarini H.S., and Qudaimat M.M., (1992) *"In-Plane Floor Deformation In RC Structures"*. ASCE Journal of Structural Engineering, Vol. 118, No. 11, pp 3089-3102.
- Schicker, V., and Powell, G.H., (1980) *"Inelastic seismic analysis of large panel buildings"*. Report No. UCB/EERC-80/38, Earthquake Engineering Research Centre, College of Engineering, University of California, Berkeley, Sept. 1980.
- Shariatmadar, H. and Mirza, M.S., (1996) *"Earthquake Resistant Connections in Precast Concrete Double-Tees"*. Proceedings of Conference on Civil Engineering by Iranian Students in Canada, Montreal, Canada, pp 259-266.
- Shariatmadar, H. and Mirza, M.S., (1995) *"An Investigation of Seismic Response of Connections in Precast Concrete Double-Tees"*. Proceedings of CSCE Annual Conference, Ottawa, Ontario, Canada, pp. 373-382.
- Shariatmadar, H. and Mirza, M.S., *"Seismic Response of Connections in Precast Concrete Double-Tee Systems"*. Paper being finalized for submission to the PCI Journal.
- Shariatmadar, H., (1992) *"An Investigation of Seismic Response of Connections in Precast concrete Double-Tees"*. M. Eng. Thesis, McGill University, Canada, 210 p.

- Shayanfar M.A., Kheyroddin, A. and Mirza, M.S., (1993) *"Finite Element Size Effect in Nonlinear Analysis of R C Structures"*. Structural Engineering Report No. 93-3, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal.
- Spencer, R.A., and Neille, P.S., (1976) *"Cyclic Tests of Welded Headed Stud Connections"*, PCI Journal, Vol. 21, No. 3, pp. 70 - 83.
- Spencer, R.A., (1986) *"Earthquake Resistant Connections for Low Rise Precast Concrete Buildings"*. Seminars on Precast Concrete Construction in Seismic Zones. Tokyo, pp. 61-81.
- Spencer, R.A., and Tong, W.K.T., (1984) *"Design of a one-story precast concrete building for earthquake loading"*. Eighth World Conference on Earthquake Engineering, Proceeding Vol. 5, pp 653-660.
- Stanton, J.F., Anderson, R.G., Donal, C.W., and McCleary, D.E., (1986) *"Design of Connections for Precast Prestressed Concrete Building for the Effect of Earthquake."* Prestressed Concrete Institute, Chicago, U.S.A. TR-5-86, 184 P.
- Stanton, J.F., (1980) *"Design Criteria for Connections in Precast Concrete"*. PCI Journal, Vol. 25, No. 2, pp. 72-92.
- Thomas, T.C. Hsu., (1968) *"Torsion of Structural Concrete - Interaction surface for Combined Torsion, Shear, and Bending in Beams without Stirrups"*, ACI Journal, Vol. 65, pp. 51-60.
- Tong, W.K.T., (1984) *"Non-Linear Seismic Analysis of a One-Storey Precast Concrete Building"*. M. A. Sc. Thesis, University of British Columbia., 240 p.
- Tsoukantas, S.G., and Tassios, T.P., (1989) *"Shear Resistance of Connection between Reinforced Concrete Linear Precast Elements"*, ACI Structural Journal, Vol. 86, No. 3, pp. 242-249.
- Ueda, T., Stitmannathum, B., and Matupayont, S., (1991) *"Experimental Investigation on Shear Strength of Bolt Anchorage Group"*, ACI Structural Journal, Vol. 88, No. 3, pp. 292-3009.
- Unemori, A.L., Roesst, J.M., and Becker, J.M., (1980) *"Effect of In-Plane Floor Slab Flexibility on the Response of Crosswalls Building Systems in Reinforced Concrete Structures Subjected to Wind and Earthquake Forces"*. ACI Publication SP-63, American Concrete Institute, Detroit, pp. 113-134
- Vintzeleou, E.N., and Tassios, T.P., (1986) *"Mathematical Models for Dowel Action under Monotonic and Cyclic Conditions"*. Magazine of Concrete Research, Vol.

38, No. 134, pp. 13-22.

Wang, M., and Shah, S.P., (1987) *"Reinforced Concrete Hysteresis Model Based on the Damage Concept"*. Journal of Earthquake Engineering and Structural Dynamics, Vol. 15, pp 993-1003.

Wilson, E.W., and Habibullah, A., (1989) *"SAP 90, A Series of Computer Programs for Static and Dynamic Finite Element Analysis of Structures"*, Computer Structures Inc., Berkeley, California, U.S.A.

Yankelevsky, D.Z., (1985) *"Bond Action Between Concrete and a Deformed Bar- A New Model"*, ACI Journal, 154-161.

APPENDIX

Design and Analysis of Connections

The ultimate strengths of the connections were calculated using the following three methods:

1. The CPCI method using a steel strength equal to its measured yield strength ($f_s = f_y$), material resistance factor for reinforcing steel, $\Phi_s = 1$, and material resistance factor for concrete, $\Phi_c = 1$, when the connection fails.
2. The modified CPCI method proposed using a steel strength equal to its measured yield strength ($f_s = f_y$), and the extra resistance at the interface of steel flange and the concrete, when the connection fails. The material resistance factors are taken as $\Phi_c = \Phi_s = 1$ for both the reinforcing steel and concrete.
3. The conceptual models, based on the resisting mechanism observed experimentally for each connection, is proposed. The pull-out strength in the tensile leg with respect to concrete cover is calculated using the equation proposed by Jimenez (1979), and the length of reinforcing bar is selected based on anchorage lengths required.

Appendix A - Shear Strength of Connections

A.1 Connection Type A

CPCI Method

Each bar leg is assumed to reach its yield strength when the connection fails. Then the ultimate compressive strength, C_u , in the compression leg and the ultimate

$$C_u = T_u = A_s \times f_y \quad (A-1)$$

tensile strength, T_u , in the tensile leg are given by:

where $A_s = 200 \text{ mm}^2$ for the M15 weldable deformed bar used

f_y = measured yield stress for steel bar = 465 MPa

Substituting in equation (A-1), gives:

$$C_u = T_u = 200 \times 465 \times 10^{-3} = 93 \text{ kN}$$

and, using the truss analogy, the value of V_{ult} is given by:

$$V_{ult} = (C_u + T_u) \cos 45^\circ \quad (A-2)$$

$$V_{ult} = \underline{131.5 \text{ kN}}$$

Modified CPCI Method

The contribution of the anchor bar is calculated when each of the reinforcing bar is assumed to reach its yield strength when the connection fails. Then the shear strength due to contribution of bar anchors, $V_{b, ult}$, using the truss analogy, are given by equation (A-2) as follows:

$$V_{b, ult} = (C_u + T_u) \cos 45^\circ \quad (A-2)$$

$$V_{b, ult} = 131.5 \text{ kN}$$

Contribution due to bond resistance at the interface of steel flange and concrete, $V_{f, ult}$, can be calculated approximately by

$$V_{f, ult} = A_f \times f'_t \quad (A-3)$$

where A_f = the area of top flange in contact with concrete = $350 \times 50 = 17500 \text{ mm}^2$

f'_t = measured ultimate tensile strength of the concrete = 3.6 MPa

Substituting these values in equation (A-3), gives:

$$V_{f, ult} = 17500 \times 3.6 \times 10^{-3} = 63 \text{ kN}$$

Using the equilibrium of the forces, the ultimate shear strength, V_{ult} is given by:

$$V_{ult} = V_{b, ult} + V_{f, ult} \quad (A-4)$$

Substituting the values in equation (A-4), gives:

$$V_{ult} = 131.5 + 63 = \underline{194.5 \text{ kN}}$$

Proposed Method

Based on the experimental observation at the ultimate load stage, equations were developed to determine the shear strength of the connection in Section 8.1.1.1. The force,

F_b , required to fail the concrete in front of compression leg is given by equation 8.1 as:

$$F_b = d_b l_e f'_c \quad (8.1)$$

where d_b is the bar diameter, l_e is the effective length of the compressive leg bearing against the concrete, found from the experimental observation to be approximately 300 mm (about $19d_b$) and the concrete compressive strength $f'_c = 43$ MPa.

Substituting in equation (8.1) gives the value of F_b as:

$$F_b = d_b l_e f'_c = 16 \times 300 \times 34 \times 10^{-3} = 163.2 \text{ kN}$$

The ultimate tensile strength in the tensile leg is calculated using equation 8.2 and by substituting the values in the equation, gives the value of T_u as:

$$T_u = \frac{d_b l_d C \sqrt{f'_c}}{424 d_b + 6.5 l_d} = \frac{16 \times 480 \times 17 \times \sqrt{34}}{424 \times 16 + 6.5 \times 480} = 76.9 \text{ kN} \quad (8.2)$$

Contribution of the steel mesh in the tension zone can be determined using equation 8.3 as following:

$$V_{sh} = n A_{sh} f_{yh} \quad (8.3)$$

where the number of the steel mesh wires crossing the crack, $n=4$,

the area of the steel mesh, $A_{sh} = 13.3 \text{ mm}^2$, and

the yield strength of steel mesh, $f_{yh} = 300 \text{ MPa}$

Substituting these values in equation (A-5) gives;

$$V_{sh} = 4 \times 13.3 \times 300 \times 10^{-3} = \underline{16 \text{ kN}}$$

Using the equilibrium of the forces in the connection, the ultimate shear strength of the connection, V_{ult} , is given by

$$V_{ult} = (F_b + T_u) \cos 45^\circ + V_{sh} \quad (8.4)$$

$$= (163.2 + 76.9) \cos 45^\circ + 16$$

$$= \underline{185.7 \text{ kN}}$$

The experimental ultimate strength of the connection AMS is 187 kN, therefore the connection capacity is predicted excellently by the proposed method.

A.2 Connection Type B

CPCI Method

Since the ultimate strength of the connection depends only on the reinforcing bar properties, the ultimate strength is the same as for the Specimen AMS.

Therefore, $V_{ult} = 131.5 \text{ kN}$

Modified CPCI Method

Since the shear strength due to contribution of the anchor bar depends only on the reinforcing bar properties, that ultimate strength is the same as for that for Specimen AMS, Therefore, $V_{b, ult} = 131.5 \text{ kN}$

Contribution due to bond resistance at the interface of steel flange and the concrete, $V_{f, ult}$, can be calculate using equation (A-3) as:

$$V_{f, ult} = A_f \times f'_t \quad (A-3)$$

where A_f = area of top flange in contact with concrete = $350 \times 50 = 17500 \text{ mm}^2$

f'_t = measured ultimate tensile strength of the concrete = 3.7 MPa

Substituting these values in equation (A-3), gives:

$$V_{f, ult} = 17500 \times 3.7 \times 10^{-3} = 64.75 \text{ kN}$$

Using the equilibrium of the forces, the ultimate shear strength, V_{ult} is given by:

$$V_{ult} = V_{b, ult} + V_{f, ult} \quad (A-4)$$

Substituting the values in equation (A-4), gives:

$$V_{ult} = 131.5 + 64.75 = 196.3 \text{ kN}$$

Proposed Method

Formulas were developed in Section 8.1.1.2 to calculate the shear strength of the connection. The contribution of the hook to the connection strength, F_{hook} is calculated using equation (8.5), and substitution of values, gives F_h as:

$$F_{hook} = \frac{1}{4} \pi d_b l_h f'_c = \frac{\pi}{4} \times 16 \times 190 \times 36 \times 10^{-3} = 85.9 \text{ kN} \quad (8.5)$$

where l_h is the length of the right angle hook part, $l_h = 12d_b = 190 \text{ mm}$, and $f'_c = 36 \text{ MPa}$

To calculate the bond strength corresponding to a splitting type of failure, the development length for the hooked bar anchor used in the test specimen was considered.

The length of straight part of the hook anchorage, l_{hd} , as proposed was,

$$l_{hd} = 400 \text{ mm}$$

The ultimate tensile strength in the hooked bar is calculated using equation (8.2),

where $l_d = l_{hb} = l_{hd} + R_{ba} + d_b = 400 + 95/2 + 16 = 432.5 \text{ mm}$

R_{ba} = inside radius of the hook

$f'_c = 36 \text{ MPa}$ (concrete compressive strength for specimen BMS)

Substituting these values in equation 8.2 gives the value of T_u , as:

$$T_u = \frac{d_b l_d C \sqrt{f'_c}}{424 d_b + 6.5 l_d} = \frac{16 \times 432.5 \times 17 \times \sqrt{36}}{424 \times 16 + 6.5 \times 432.5} = 73.6 \text{ kN}$$

The force required to push off the concrete in the compression bar leg is calculated using equation (8.1). The effective length bearing against the concrete was noted to be about $17d_b = 270 \text{ mm}$, which gives the strength F_b as:

$$F_b = d_b l_c f'_c = 16 \times 270 \times 36 \times 10^{-3} = 155.5 \text{ kN}$$

Contribution of the steel mesh in the tension zone can be determined as:

$$V_{sh} = n A_{sh} f_{yh} = 3 \times 13.3 \times 300 \times 10^{-3} = \underline{12 \text{ kN}} \quad (8.3)$$

where the number of the steel mesh crossing the crack, $n=3$

the cross-sectional area of the steel mesh, $A_{sh} = 13.3 \text{ mm}^2$

the yield strength of steel mesh, $f_{yh} = 300 \text{ MPa}$

Writing the equation for equilibrium of the forces for the connection, the ultimate strength of Specimen BMS, V_{ult} , is given by

$$\begin{aligned} V_{ult} &= (F_h + T_u + F_b) \cos 45^\circ + V_{sh} \\ &= (85.9 + 73.6 + 155.5) \cos 45^\circ + 12 \\ &= \underline{234.7 \text{ kN}} \end{aligned} \quad (8.6)$$

The experimental ultimate strength of the connection is 242 kN, therefore the connection strength is estimated excellently by the proposed method (experimental strength is just about 3 percent more than the predicted strength).

A.3 Connection Type C

CPCI Method

The reinforcement bars welded to the angle should be large enough to ensure that they do not yield even when the connecting plate (welded to the plate or angles on either side of joint) is stressed to its ultimate strength. This minimizes the cracking damage to the panel. It should be noted that the material resistance factors Φ_c and Φ_s were considered to be equal to 1 to obtain the unfactored strength of the connection.

The connection works by the shear-friction mechanism, and Clause 11.7.4 to 11.7.10 of CSA Standard CAN3-A23.3-M84 are used to calculate the ultimate strength of the Specimen CMS as follows:

Since the reinforcing bars are perpendicular to the shear plane, the ultimate unfactored shear-resistance V_{ult} , is calculated using the following equation:

$$V_{ult} = A_{vf} f_y \mu \quad (A-5)$$

where A_{vf} = area of shear-friction reinforcing bar = $2 \times 200 = 400 \text{ mm}^2$

$\mu = 0.8\lambda$ for concrete anchored to steel plate by the reinforcing bar and the contact plane one full plate thickness below the concrete surface

$\lambda = 1$ for normal density concrete

f_y = yield strength of the reinforcing bar, but shall not be greater than 400 MPa.

Substituting values in equation (A-5), gives

$$V_{ult} = A_{vf} f_y \mu = 400 \times 400 \times 0.8 \times 1 \times 10^{-3} = \underline{128 \text{ kN}}$$

The shear strength is checked using Clause 11.7.5, according to which, the ultimate strength must not be greater than

$$V_{ult} = 0.25 f'_c A_{cv} \quad (A-6)$$

where $f'_c = 43 \text{ MPa}$ (compressive strength of the concrete)

A_{cv} = area of the concrete section resisting shear transfer
 $= 200 \times (75 + 50) = 25,000 \text{ mm}^2$

Substituting the values in equation (A-6), gives:

$$V_{ult} = 0.25 \times 43 \times 25000 \times 10^{-3} = 268.75 \text{ kN}$$

Also, the strength obtained by equation (A-5) shall not be greater than the value given by

$$V_{ult} = 6.5 A_{cv} = 6.5 \times 25000 \times 10^{-3} = 162.5 \text{ kN} \quad (\text{A-7})$$

Therefore, the ultimate shear strength, V_{ult} , of Specimen CMS is

$$V_{ult} = \underline{128 \text{ MPa}}$$

Modified CPCI Method

Since the reinforcing bars are perpendicular to the shear plane, and under go the actual yield strain, then when using shear-friction to calculate the shear strength of the connection utilizing the anchor bar measured yield strength instead of nominal strength (as suggested by CPCI not to be greater than 400 MPa) is a logical assumption. Therefore, the ultimate unfactored shear-resistance V_{ult} , is calculated using the following equation:

$$V_{ult} = A_{vf} f_y \mu \quad (\text{A-5})$$

where the terms are those used in CPCI method, except that $f_y = 465 \text{ MPa}$ is the actual yield strength of the reinforcing bar measured experimentally.

Substituting values in equation (A-5), gives

$$V_{ult} = A_{vf} f_y \mu = 400 \times 465 \times 0.8 \times 1 \times 10^{-3} = \underline{148.8 \text{ kN}}$$

Proposed Method

As described in Section 8.1.1.3, the lateral bearing of the bar anchor which act on partial bar length as a stud anchor, controls the resisting mechanism. The bearing resistance of the bar anchor is calculated using equation 8.7 as:

$$F_b = 1.4 d_b l_e f'_c \quad (8.7)$$

It is shown that $5d_b \leq l_e \leq 8d_b$ (Marcus 1951). For the connection the value of l_e was found to be equal to about $5.5d_b = 90 \text{ mm}$. The shear strength of the Connection Type C with two anchor bars can be taken as:

$$V_{ult} = 2 \times 1.4 \times 5.5 \times d_b^2 f'_c \quad (8.8)$$

Substituting the values in the above equation, gives the ultimate shear strength of Connection Type C as follows:

$$V_{ult} = 2 \times 1.4 \times 5.5 \times 16^2 \times 38 \times 10^{-3} = \underline{149.8 \text{ kN}}$$

The experimental ultimate strength of the connection CMS is 152 kN, thus the connection strength is estimated excellently by proposed method.

A.4 Connection Type D

CPCI Method

The ultimate strength of the connection can be calculated by using the shear friction concept. The factored shear strength, V_{ult} of the bar perpendicular to the shear plane is calculated using equation (A-5):

$$V_{ult} = A_{vf} f_y \mu = 200 \times 400 \times 0.8 \times 1 \times 10^{-3} = 64 \text{ kN} \quad (A-5)$$

The shear strength is checked using Clause 11.7.5, according to which, the ultimate strength must not be greater than

$$V_{ult} = 0.25 f'_c A_{cv} \quad (A-6)$$

where $f'_c = 41 \text{ MPa}$ (compressive strength of the concrete)

$$\begin{aligned} A_{cv} &= \text{area of the concrete section resisting shear transfer} \\ &= 250 \times (75 + 50) = 31,250 \text{ mm}^2 \end{aligned}$$

Substituting the values in equation (A-6), gives:

$$V_{ult} = 0.25 \times 41 \times 31250 \times 10^{-3} = 320 \text{ kN}$$

Also, the strength obtained by equation (A-5), shall not be greater than value given by

$$V_{ult} = 6.5 A_{cv} = 6.5 \times 31250 \times 10^{-3} = 203.12 \text{ kN} \quad (A-7)$$

Therefore, the ultimate strength of the reinforcing bar perpendicular to the shear plane is,

$$V_{ult} = \underline{64 \text{ kN}}$$

The factored shear resistance of the bars inclined to the shear plane at an angle α , can be

computed using the following equation suggested in the CSA Standard CAN-A23.3-M84

$$V_r = A_{vf} f_y (\mu \sin \alpha + \cos \alpha) \quad (A-8)$$

where A_{vf} = Area of shear friction reinforcing bar = $2 \times 200 = 400 \text{ mm}^2$

$\mu = 0.8 \lambda$ for the concrete anchored to the steel plate by the reinforcing bar and the contact plane one full plate thickness below the concrete surface.

$\lambda = 1$ for normal density concrete.

f_y = yield strength of the bar anchor, but shall not be greater than 400 MPa.

Substituting these values in equation (A-8), gives

$$V_{ult} = 400 \times 400 \times (0.8 \sin 45^\circ + \cos 45^\circ) \times 10^{-3} = 203.6 \text{ kN}$$

Therefore, V_{ult} for the connection DMS is,

$$V_{ult} = 64 + 203.6 = \underline{267.6 \text{ kN}}$$

The shear strength is checked using Clause 11.7.5, according to which, the ultimate strength must not be greater than

$$V_{ult} = 0.25 f'_c A_{cv} \quad (A-6)$$

where $f'_c = 42 \text{ MPa}$ (compressive strength of the concrete)

$$\begin{aligned} A_{cv} &= \text{area of the concrete section resisting shear transfer} \\ &= 250 \times (75 + 50) = 31250 \text{ mm}^2 \end{aligned}$$

Substituting these values in equation (A-6), gives:

$$V_{ult} = 0.25 \times 42 \times 31250 \times 10^{-3} = 328 \text{ kN}$$

Also, the strength obtained by equation (A-7), shall not be greater than value given by:

$$\begin{aligned} V_{ult} &= 6.5 A_{cv} \\ &= 6.5 \times 31250 \times 10^{-3} = 203.12 \text{ kN} \end{aligned}$$

Therefore, the ultimate strength of the connection DMS is:

$$V_{ult} = \underline{203.12 \text{ kN}}$$

Proposed Method

As described in Section 8.1.14, three components contribute to the ultimate strength of the connection: strength of the diagonal shear anchor bar crossing the

diagonal crack, V_{dh} , strength due to the horizontal steel mesh crossing the diagonal crack, V_{sh} , and the strength contribution of the shear resistance of the concrete surrounding the connection anchors, V_{ch} . The ultimate strength of the connection, V_{ult} is calculated as:

$$V_{ult} = V_{dh} + V_{sh} + V_{ch} \quad (8.9)$$

The strength due to tensile anchor bar, V_{dh} , is given by

$$\begin{aligned} V_{dh} &= A_{sd} f_{yd} \cos 45^\circ \\ &= 200 \times 465 \times \cos 45^\circ \times 10^{-3} = \underline{65 \text{ kN}} \end{aligned} \quad (8.10)$$

Contribution of the steel mesh, V_{sh} , can be found by

$$V_{sh} = n A_{sh} f_{yh} \quad (8.3)$$

where the number of the steel mesh wires crossing the crack, $n=8$

the area of the steel mesh, $A_{sh} = 13.3 \text{ mm}^2$

yield strength of steel mesh, $f_{yh} = 300 \text{ MPa}$

Substituting these values in equation (A-19) gives;

$$V_{sh} = 8 \times 13.3 \times 300 \times 10^{-3} = \underline{31.9 \text{ kN}}$$

The contribution of the concrete to the shear resistance, V_{ch} which act as the "shear-cone" can be calculated by:

$$V_{ch} = 0.3 A \sqrt{f'_c} \quad (8.11)$$

where the effective stress area, A , can be calculated by: $A = L_w t - 3 A_s$, in which, the thickness of the concrete panel, $t=50 \text{ mm}$, $A_s=200 \text{ mm}^2$ is the area of one bar anchor, and, $L_w = 2 \times (l_d / \cos 45^\circ) + 100 = 1457 \leq 1300 \text{ mm}$, therefore $L_w = 1300 \text{ mm}$. Then the effective stress area is $A = 1300 \times 50 - 3 \times 200 = 64400 \text{ mm}^2$.

Substituting these values in equation (A-20) gives:

$$V_{ch} = 0.3 \times 64400 \times \sqrt{41} \times 10^{-3} = \underline{123.7 \text{ kN}}$$

Therefore, the ultimate shear strength of the Connection Type D is calculated by substituting the values of these contributing strengths in equation (8.9) as follows;

$$V_{ult} = V_{dh} + V_{sh} + V_{ch} = 65.7 + 31.9 + 123.7 = \underline{221.3 \text{ kN}}$$

The experimental ultimate strength for the Specimen DMS is 226 kN, therefore the predicted connection strength using the proposed method is in excellent agreement with

the experimental results.

A.5 Connection Type E

CPCI Method

When the studs are welded to the plate, the ultimate strength of the connection, V_{ult} , is calculated using shear friction principles. If the connection fails because of exhaustion of the stud strength, the connection strength is calculated as :

$$V_{ult} = A_{vf} f_y \mu \quad (A-5)$$

where A_{vf} = cross sectional area of the studs = $2 \times 71 = 142 \text{ mm}^2$

$\mu = 0.8\lambda$ for the contact plane, a full plate thickness below the concrete surface

$\lambda = 1$ for normal density concrete

f_y = yield strength of the studs = 400 MPa

Substituting these values in equation (A-5), gives:

$$V_{ult} = A_{vf} f_y \mu = 142 \times 400 \times 0.8 \times 1 \times 10^{-3} = 45.4 \text{ kN}$$

The shear strength is checked using Section 4.5 of the CPCI Metric Design Manual (CPCI, 1987), according to which, the ultimate strength should be less than

$$V_{ult} = 0.25 f'_c A_{cv} \quad (A-6)$$

where $f'_c = 40 \text{ MPa}$ (compressive strength of the concrete)

A_{cv} = area of the concrete section resisting shear transfer

$$= 150 \times 50 = 7500 \text{ mm}^2$$

Substituting these values in equation (A-11), gives:

$$V_{ult} = 0.25 \times 40 \times 7500 \times 10^{-3} = 75 \text{ kN}$$

Also, the strength obtained by equation (A-5) shall not be greater than value given by

$$V_{ult} = 6.5 A_{cv} = 6.5 \times 7500 \times 10^{-3} = 48.75 \text{ kN} \quad (A-7)$$

Therefore, the ultimate shear strength, V_{ult} , of the Specimen EMS is

$$V_{ult} = \underline{45.4 \text{ MPa}}$$

Proposed Method

Based on the discussion in Section 8.1.1.5, the ultimate tensile strength of the steel anchor should be utilized instead of the yield strength, when the shear friction concept is used to predict the ultimate strength of the connection.

The ultimate shear strength can be calculated using equation (8.12), as followings:

$$V_{ult} = A_{vf} f_{ult} \mu \quad (8.12)$$

where A_{vf} = cross sectional area of the studs = $2 \times 71 = 142 \text{ mm}^2$

$\mu = 0.8\lambda$ for contact plane a full plate thickness below the concrete surface

$\lambda = 1$ for normal density concrete

f_{ult} = ultimate strength of the studs = 435 MPa

Substituting the values in equation (8.12), gives the shear strength of Connection Type E, V_{ult} as:

$$V_{ult} = 142 \times 435 \times 0.8 \times 10^{-3} = \underline{49.4 \text{ kN}}$$

The experimental ultimate strength for connection EMS is 52.6 kN. Therefore the predicted strength of the connection by the proposed method is in excellent agreement with the experimental result.

Summary

The CPCI methods give conservative results for all the connections subjected to shearing loads. The analytical ultimate strength calculated by the proposed methods give very good to excellent agreement with the experimental results for all connection types. Therefore, the theoretical strength of the connections under static loads can be found accurately using the conceptual methods proposed in the present study.

Appendix B -Axial (Tensile) Strength of Connections

B.1 Connection Type A

CPCI Method

Each bar leg is assumed to reach its yield strength when the connection fails. Then the ultimate tensile strength, T_u , for both tensile legs is given by:

$$T_u = A_s \times f_y \quad (A-1)$$

where $A_s = 200 \text{ mm}^2$

f_y = the measured yield strength for steel bar = 465 MPa

Substituting these values in equation (A-1), gives:

$$T_u = 200 \times 465 \times 10^{-3} = 93 \text{ kN}$$

and, using the truss analogy, the value of V_{ult} is given by:

$$T_{ult} = 2 T_u \cos 45^\circ \quad (A-9)$$

$$T_{ult} = \underline{131.5 \text{ kN}}$$

Proposed Method

According to Section 8.1.2.1, the ultimate strength of the connection should be calculated based on splitting bond strength. The ultimate tensile strengths of the both tensile legs are identical, as follows:

$$T_u = \frac{d_b l_d C \sqrt{f'_c}}{424 d_b + 6.5 l_d} = \frac{16 \times 480 \times 17 \times \sqrt{35}}{424 \times 16 + 6.5 \times 480} = 77.9 \text{ kN} \quad (8.2)$$

Writing the force equilibrium condition for the joint using the "truss analogy", the ultimate axial (pull-out) strength of the connection, T_{ult} , is given by:

$$T_{ult} = 2 T_u \cos 45^\circ \quad (A-9)$$

$$= 2 \times 77.9 \times \cos 45^\circ = \underline{110.2 \text{ kN}}$$

The experimental ultimate strength of the connection AT under cyclic loading is 106 kN.

Therefore, the proposed method can predicted the ultimate capacity of the connection very closely.

B.2 Connection Type B

CPCI Method

Since the ultimate strength of the connection depends only on the reinforcing bar properties, the ultimate strength is the same as for Specimen AT.

Therefore,

$$V_{ult} = \underline{131.5 \text{ kN}}$$

Proposed Method

According to Section 8.1.2.2, the ultimate strength is achieved, when the concrete in front of the hooks pushed off by crushing and steel mesh crossing the cracks yielded.

The contribution of each hook in the connection strength, T_{hook} is :

$$T_{hook} = \frac{\pi}{4} d_b l_h f'_c = \frac{\pi}{4} \times 16 \times 190 \times 36 \times 10^{-3} = 85.9 \text{ kN} \quad (8.14)$$

Where l_h is the length of right angle part of the hook, $l_h = 12d_b = 190 \text{ mm}$, and $f'_c = 36 \text{ MPa}$

Contribution of the steel mesh in the tension zone can be determined as follows:

$$T_{sv} = n A_{sv} f_{yv} \quad (8.15)$$

where the number of the steel mesh wires crossing the cracks, $n=9$

the area of the steel mesh, $A_{sv} = 13.3 \text{ mm}^2$

the yield strength of the steel mesh, $f_{yv} = 300 \text{ MPa}$

Substituting these values in the equation (8.15) gives;

$$T_{sv} = 9 \times 13.3 \times 300 \times 10^{-3} = \underline{35.9 \text{ kN}}$$

Writing the equation for equilibrium of the forces in the axial direction for both the hooks and the resistance due to the steel mesh, and substituting the values, the ultimate strength of Specimen BT, T_{ult} , is calculated as following:

$$T_{ult} = 2 \times T_{hook} \times \cos 45^\circ + T_{sv} = 2 \times 85.9 \times \cos 45^\circ + 35.9 = \underline{157.4 \text{ kN}} \quad (8.16)$$

The predicted ultimate strength using the proposed method agrees very well with the experimental ultimate strength of the Specimen BT (150 kN).

B.3 Connection Type C

CPCI Method

Each bar leg is assumed to reach its yield strength when the connection fails. Then the ultimate tensile strength, T_u , due to both tensile legs is given by:

$$T_u = A_s \times f_y \quad (A-1)$$

where $A_s = 200 \text{ mm}^2$

f_y = the measured yield strength for the steel bar = 465 MPa

Substituting these values in equation (A-1), gives:

$$T_u = 200 \times 465 \times 10^{-3} = 93 \text{ kN}$$

and, writing the force equilibrium in the axial direction, the value of T_{ult} is given by:

$$T_{ult} = 2 T_u \quad (A-10)$$

Therefore,

$$T_{ult} = \underline{186 \text{ kN}}$$

Proposed Method

According to Section 8.1.2.3, equation (8.2) can be used to determine the ultimate strength of each leg, as follows:

$$T_u = \frac{d_b l_d C \sqrt{f'_c}}{424 d_b + 6.5 l_d} = \frac{16 \times 480 \times 17 \times \sqrt{38}}{424 \times 16 + 6.5 \times 480} = 81.3 \text{ kN} \quad (8.2)$$

The ultimate axial strength of Connection Type C, with two anchor bars can be taken as:

$$\begin{aligned} T_{ult} &= 2 T_u \\ &= 2 \times 81.3 = \underline{162.6 \text{ kN}} \end{aligned} \quad (A-10)$$

The experimental ultimate strength of the Specimen CT is 150 kN, which is 8% lower than the analytical value due to the cyclic axial loading associated with the spalling of the concrete.

B.4 Connection Type D

CPCI Method

Each bar leg is assumed to reach its yield strength when the connection fails. Then the ultimate tensile strength, T_u , in each legs is given by:

$$T_u = A_s \times f_y \quad (A-1)$$

where $A_s = 200 \text{ mm}^2$

f_y = the measured yield strength for the steel bar = 465 MPa

Substituting these values in equation (A-1), gives:

$$T_u = 200 \times 465 \times 10^{-3} = 93 \text{ kN}$$

and, writing the equation for the force equilibrium in the axial direction, the value of T_{ult} is given by:

$$T_{ult} = (2 \times \cos 45^\circ + 1) T_u \quad (A-11)$$

Therefore,

$$T_{ult} = \underline{224.5 \text{ kN}}$$

Proposed Method

According to Section 8.1.2.4, the strength of the connection can be determined utilizing the concept of the tensile "shear-cone" failure and accounting for the steel mesh crossing the conical surface (diagonal cracks formed at ends of all the three bars).

The contribution of the concrete to the pull-out resistance, T_{cv} which act as "shear-cone" can be calculated as follows:

$$T_{cv} = 0.3 A \sqrt{f'_c} \quad (8.18)$$

where the effective stress area, $A = L_w t - 3 A_s$,

where the thickness of the concrete panel, $t = 50 \text{ mm}$, $A_s = 200 \text{ mm}^2$, the area of one bar anchor, and $L_w = 2(l_d / \cos 45^\circ) + 100 = 1457 \leq 1300 \text{ mm}$, therefore, $L_w = 1300 \text{ mm}$.

Then the effective stress area is $A = 1300 \times 50 - 3 \times 200 = 64400 \text{ mm}^2$

Substituting the values in equation (8.18) gives:

$$V_{ch} = 0.3 \times 64400 \times \sqrt{41} \times 10^{-3} = \underline{123.7 \text{ kN}}$$

Contribution of the steel mesh crossing conical surface can be determined as following:

$$T_{sv} = n A_{vh} f_{yv} \quad (8.15)$$

where the number of the steel mesh wires crossing the cone vertically, $n=13$

the area of the steel mesh wires, $A_{vh} = 13.3 \text{ mm}^2$

the yield strength of steel mesh, $f_{yv} = 300 \text{ MPa}$

Substituting these values in the equation (8.15) gives;

$$T_{vh} = 13 \times 13.3 \times 300 \times 10^{-3} = \underline{51.9 \text{ kN}}$$

Writing the equation of the forces equilibrium in the axial direction for the "shear-cone" resistance and the resistance of the steel mesh, the ultimate strength of Specimen DT, T_{ult} , is given by substituting the values of these contributing strengths in equation (8.19) as:

$$T_{ult} = T_{cv} + T_{sv} \quad (8.19)$$

$$= 123.7 + 51.9$$

$$T_{ult} = \underline{175.6 \text{ kN}}$$

The ultimate experimental strength is just about 2% higher than the predicted value using the proposed method, showing excellent agreement. Therefore, the strength of the connection can be found using the proposed method present herein.

B.5 Connection Type E

CPCI Method

The tensile capacity of the concrete surrounding a headed stud, for failure by a cone-type pull-out, $T_{c, ult}$, is given by:

$$T_{c, ult} = 0.3 A \sqrt{f'_c} \quad (8.18)$$

where the effective stress area, A is calculated as:

$$\begin{aligned} A &= (2l_e + 75 + 1.58d_b)t - 2 \times 2.5 d_b^2 \\ &= (2 \times 100 + 75 + 1.58 \times 10) \times 50 - 2 \times 2.5 \times 10^2 = \underline{14040 \text{ mm}^2} \end{aligned}$$

Substituting these values in equation (8.18) gives, $T_{c, ult}$ as:

$$T_{c, ult} = 0.3 \times 14090 \times \sqrt{40} \times 10^{-3} = 26.7 \text{ kN}$$

The ultimate resistance of the connection as governed by the tensile strength of the stud material is calculated by:

$$T_{s, ult} = A_b f_u \quad (A-12)$$

where the cross-sectional area of the stud anchors, $A_b = 2 \times 71 = 142 \text{ mm}^2$

the ultimate strength of the anchor material, $f_u = 435$.

Substituting these values in equation (A-28), gives:

$$T_{s, ult} = 142 \times 435 \times 10^{-3} = 61.7 \text{ kN}$$

Therefore, the ultimate tensile strength, T_{ult} , of the Connection Type E is

$$T_{ult} = \underline{26.7 \text{ kN}}$$

Proposed Method

As discussed in Section 8.1.2.5, the ultimate tensile strength of Connection Type E, T_{ult} , can be calculated by the following equation:

$$T_{ult} = n P l f'_t \quad (8.20)$$

where the perimeter of the stud head, $P = \pi d_h = \pi \times 16 = 50.3 \text{ mm}^2$

the embedment length, $l = 100 \text{ mm}$

the number of the studs, $n = 2$

the concrete tensile strength, $f'_t = 3.8 \text{ MPa}$

Substituting these values in the equation (8.20), gives

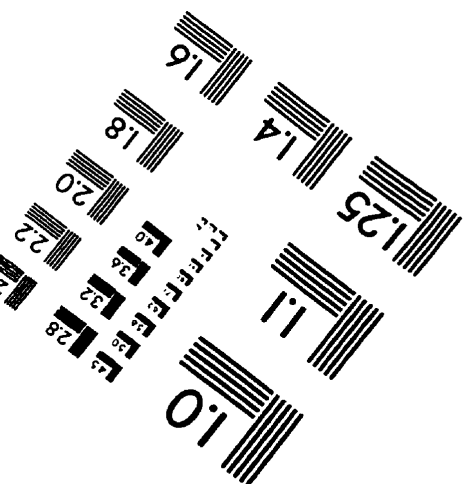
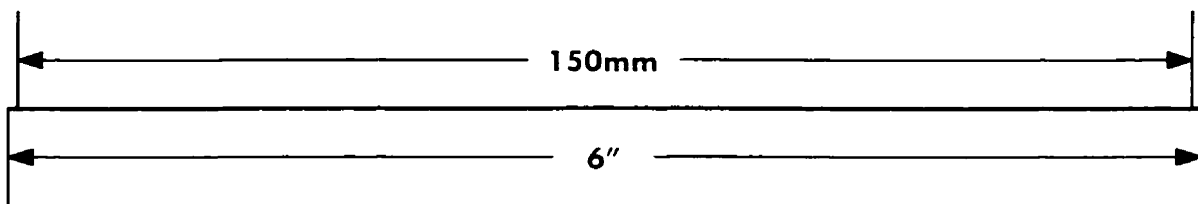
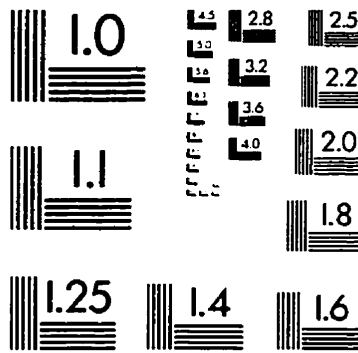
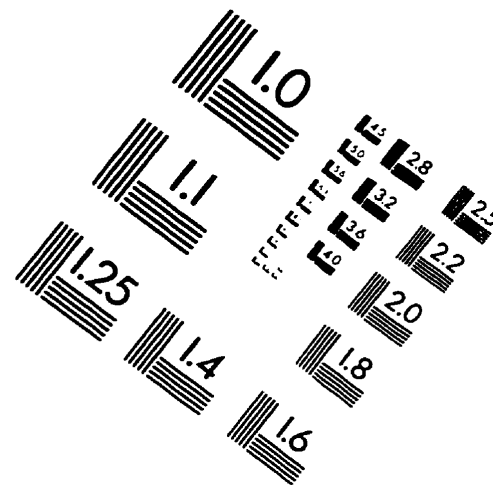
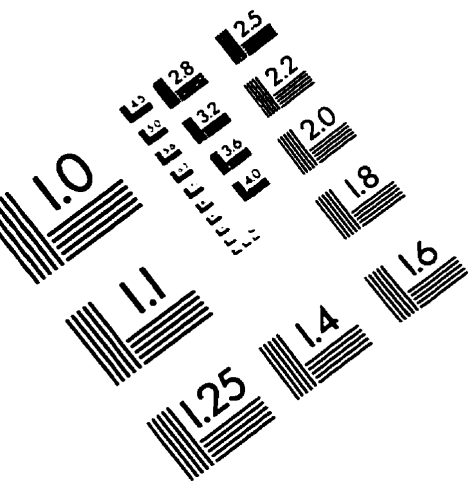
$$T_{ult} = 2 \times 50.3 \times 100 \times 3.8 \times 10^{-3} = \underline{38.3 \text{ kN}}$$

The experimental ultimate strength is 32 kN, which is smaller than the analytical strength due to cyclic loading. However, the method presented herein can be use to found the nominal strength of the connection with reasonable accuracy.

Summary

The experimental strength of the connections under cyclic axial load are smaller than the ultimate strength calculated using the CPCI methods, except for Specimens BT and ET for which the CPCI methods give conservative results. The analytical ultimate strength calculated by the proposed methods give very good to excellent agreement with the experimental results for all connection types. Therefore, the theoretical strength of the connections under static loads can be found accurately using the conceptual methods proposed in the present study.

IMAGE EVALUATION TEST TARGET (QA-3)



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