1

A

07544

National Library
of Canada

Bibliothèque nationale
du Canada

CANADIAN THESES
ON MICROFICHE

THÈSES CANADIENNES
SUR MICROFICHE

NAME OF AUTHOR/NOM DE L'AUTEUR KENG-CHEE HONG

TITLE OF THESIS/TITRE DE LA THÈSE "Behaviour of Composite Beam-to-Column Joint under
Cyclic Loading"

UNIVERSITY/UNIVERSITÉ Carleton University

DEGREE FOR WHICH THESIS WAS PRESENTED/
GRADE POUR LEQUEL CETTE THÈSE FUT PRÉSENTÉE M. Eng.

YEAR THIS DEGREE CONFERRED/ANNÉE D'OBTENTION DE CE DEGRÉ 1978

NAME OF SUPERVISOR/NOM DU DIRECTEUR DE THÈSE J. L. Humar

Permission is hereby granted to the NATIONAL LIBRARY OF CANADA to microfilm this thesis and to lend or sell copies of the film.

L'autorisation est, par la présente, accordée à la BIBLIOTHÈQUE NATIONALE DU CANADA de microfilmer cette thèse et de prêter ou de vendre des exemplaires du film.

The author reserves other publication rights, and neither the thesis nor extensive extracts from it may be printed or otherwise reproduced without the author's written permission.

L'auteur se réserve les autres droits de publication, ni la thèse ni de longs extraits de celle-ci ne doivent être imprimés ou autrement reproduits sans l'autorisation écrite de l'auteur.

DATED/DATE March 16 1978 SIGNED/SIGNÉ

PERMANENT ADDRESS/RÉSIDENCE FIXE _____

National Library of Canada

Cataloguing Branch
Canadian Theses Division

Ottawa, Canada
K1A 0N4

Bibliothèque nationale du Canada

Direction du catalogage
Division des thèses canadiennes

NOTICE

The quality of this microfiche is heavily dependent upon the quality of the original thesis submitted for microfilming. Every effort has been made to ensure the highest quality of reproduction possible.

If pages are missing, contact the university which granted the degree.

Some pages may have indistinct print especially if the original pages were typed with a poor typewriter ribbon or if the university sent us a poor photocopy.

Previously copyrighted materials (journal articles, published tests, etc.) are not filmed.

Reproduction in full or in part of this film is governed by the Canadian Copyright Act, R.S.C. 1970, c. C-30. Please read the authorization forms which accompany this thesis.

**THIS DISSERTATION
HAS BEEN MICROFILMED
EXACTLY AS RECEIVED**

AVIS

La qualité de cette microfiche dépend grandement de la qualité de la thèse soumise au microfilmage. Nous avons tout fait pour assurer une qualité supérieure de reproduction.

Si il manque des pages, veuillez communiquer avec l'université qui a conféré le grade.

La qualité d'impression de certaines pages peut laisser à désirer, surtout si les pages originales ont été dactylographiées à l'aide d'un ruban usé ou si l'université nous a fait parvenir une photocopie de mauvaise qualité.

Les documents qui font déjà l'objet d'un droit d'auteur (articles de revue, examens publiés, etc.) ne sont pas microfilmés.

La reproduction, même partielle, de ce microfilm est soumise à la Loi canadienne sur le droit d'auteur, SRC 1970, c. C-30. Veuillez prendre connaissance des formules d'autorisation qui accompagnent cette thèse.

**LA THÈSE A ÉTÉ
MICROFILMÉE TELLE QUE
NOUS L'AVONS REÇUE**

**BEHAVIOR OF COMPOSITE
BEAM-TO-COLUMN JOINT
UNDER CYCLIC LOADING**

by K. C. HONG

MASTERS THESIS

DEPARTMENT OF CIVIL ENGINEERING

CARLETON UNIVERSITY

OTTAWA, CANADA K1S 5B6

JANUARY 1978

The undersigned recommend to the Faculty of
Graduate Studies acceptance of the thesis:

"BEHAVIOR OF COMPOSITE BEAM-TO-COLUMN
JOINT UNDER CYCLIC LOADING"

submitted by Keng Chee Hong, B.Sc.(Eng), in
partial fulfillment of the requirement of the
degree of

MASTER OF ENGINEERING

Thesis Supervisor

Chairman, Department of
Civil Engineering

Carleton University

BEHAVIOR OF COMPOSITE BEAM-TO-COLUMN
JOINT UNDER CYCLIC LOADING

by

KENG CHEE HONG

B.Sc. (Eng)

A THESIS

Presented to the Faculty of Graduate Studies
of Carleton University in partial
fulfillment of the degree of
MASTER OF ENGINEERING IN CIVIL ENGINEERING

Carleton University

Ottawa, Canada

January, 1978

ABSTRACT

The objective of this thesis was to estimate, by experiment, the ductility of a beam-to-column joint, in which the beam had a composite steel-concrete section and the column was of steel, under cyclic loading. Three specimens were tested. The first of the specimens was a plain steel beam-to-column joint, the other two had steel column stubs and steel-concrete composite beams simulating an interior beam-to-column joint of a steel building frame with concrete floor.

A compact steel section was used for the beam of all the three specimens to minimize the possibility of premature failure due to local instability of web and flange at the joint. Horizontal web stiffeners were installed for the third specimen to examine the effect the stiffeners have on the ductility of the joint. The specimens were loaded in a quasi-static manner by using a hydraulically powered, electronically controlled actuator. The moment-curvature, moment-rotation and load-deflection relationships were obtained and were used to obtain estimates of the ductility.

Premature failure due to local buckling did not occur, all specimens failed at a load higher than or equal to their corresponding plastic loads under negative bending. The homogeneous steel section and the composite steel-concrete sections exhibited stable hysteresis loops under cyclic load-

ing. They also showed good ductility capacity. The ductility under negative bending was considerably improved when the beam web was strengthened by welding of longitudinal plate stiffeners on the web.

ACKNOWLEDGEMENTS

The author wishes to express his gratitude to Prof. J.L. Humar for his encouragement and advice throughout the course of this study.

The assistance of the staff of the structural laboratory; B. Davies and K. McMartin, is sincerely appreciated. Thanks are due to R.A. Hill and J. Herler of the mechanical workshop for their help in the fabrication of the test components.

Financial support provided for this study by the National Research Council of Canada is acknowledged.

Sincere thanks are extended to Mrs. I.J. Hong for her assistance in preparing the figures and typing the manuscript.

2

TABLE OF CONTENTS

	Page
ABSTRACT	i
ACKNOWLEDGEMENTS	iii
TABLE OF CONTENTS	iv
NOTATION	vi
 CHAPTER I INTRODUCTION	
1.1 Introductory remarks	1
1.2 Review of previous research	3
1.3 Scope of present study	11
 CHAPTER II TEST PROGRAM	
2.1 Test specimens	30
2.2 Test apparatus	35
2.3 Instrumentation	39
2.4 Material properties	42
2.5 Fabrication of test specimens	43
2.6 Test procedure	45
 CHAPTER III TEST RESULTS	
3.1 Introduction	88
3.2 General behavior	88
3.3 Specimen SB1	90
3.4 Specimen SB2	92
3.5 Specimen SB3	94
3.6 Strain in longitudinal reinforcement	96
3.7 Cracking of concrete slab	96

	Page
3.8 Ultimate load conditions	97
 CHAPTER IV DISCUSSION OF TEST RESULTS	
4.1 Introduction	124
4.2 Specimen SB1	124
4.3 Specimen SB2	128
4.4 Specimen SB3	130
4.5 General behavior	133
4.6 Strain in longitudinal reinforcement	136
 CHAPTER V	
5.1 Summary	142
5.2 Conclusions	142
5.3 Recommendations	144
 LIST OF REFERENCES	 148
 APPENDIX A DESIGN CALCULATIONS	 A1
APPENDIX B PERFORMANCE DATA OF TEST MACHINE	B1
APPENDIX C CALCULATION OF ACTUAL PLASTIC MOMENT AND ROTATION	 C1

NOTATION

A_r = area of longitudinal slab reinforcement.
 A_s = area of steel beam.
 a = position of neutral axis (plastic).
 b = half the width of the flange of steel beam.
 b_1 = width of concrete slab.
 C' = deflection coefficient.
 d = distance from top of concrete slab to the centroid of steel beam.
 d' = depth of steel beam.
 d_r = distance from top of concrete slab to the centre of longitudinal reinforcement.
 E_s = modulus of elasticity of steel.
 E_c = modulus of elasticity of concrete.
 F_y = yield stress of steel beam.
 f_c' = specified strength of concrete in compression.
 f_y' = yield stress of longitudinal reinforcement.
 h = clear depth of web of steel beam.
 I = moment of inertia in positive bending.
 I' = moment of inertia in negative bending.
 I_s = moment of inertia of steel beam.
 l_p = distance from centre of compression concrete to the centroid of steel beam.
 l = distance from the centre of roller support to the face of the column stub.

L_1 = distance between the two roller supports.

M_p = plastic moment capacity of composite section (positive bending).

M_p' = plastic moment capacity of composite section (negative bending).

M_{ps} = plastic moment of steel beam.

M_1 = applied moment at 3" from the face of the column stub.

M_2 = applied moment at the face of the column stub.

P = applied load.

t = flange thickness of steel beam.

V_n = force on shear connectors.

w = web thickness of steel beam.

W = density of concrete.

x = position of neutral axis (plastic).

x_1 = position of neutral axis (elastic).

ϕ = curvature at 3" from the face of the column.

θ = total end rotation.

θ_p = total end rotation at yield.

Δ = central deflection.

CHAPTER I

INTRODUCTION

1.1 INTRODUCTORY REMARKS

In the structural design of multistory building frames located in seismic zones, two different design objectives can be aimed for - one in which the structure is designed to remain elastic, and the other in which it is permitted to become inelastic during a severe earthquake.

The basis of the first approach is to design the structure to remain elastic, i.e. to ensure that no permanent deformation occurs even when the structure is subjected to the most severe earthquake that it is likely to experience. The stiffness of the building designed according to this approach will remain unchanged throughout the period of an earthquake. This in reality would give a very rigid building structure. However, the cost of such form of construction does not justify against the economic and social consequences that are valued upon the probability of occurrence of an earthquake.

On the other hand, the second approach allows inelastic deformations to take place in the building structure when it is subjected to high intensity earthquake forces. Studies of single and multi-degree of freedom systems subjected to earthquake excitation [1] * have shown that for a building frame

* : Numbers in square brackets refer to the references listed at the end of thesis.

the elastic and inelastic spectral displacements are the same. This is illustrated in the simplified displacement diagram Figure 1.1, which shows how a low yield-point structural system relates to an elastic system when both undergo the same displacement.

The capacity of a low-yield-point structural system to undergo inelastic deformations without impairing its load carrying capacity is defined as the Ductility Capacity of the system. In mathematical term, this property is defined by a parameter called Ductility Ratio, which is the ratio of maximum deformation to the deformation at yield as indicated in Figure 1.1. The higher the ductility ratio, the larger the inelastic deformation the structure can sustain. The capacity for inelastic deformation permits one to design the structure for a force level which is only a fraction of the forces computed on the assumption that the given structure remains elastic during the earthquake. From Figure 1.1 it can be seen that the low level of forces are equal to the elastic forces divided by the ductility ratio.

In reality, inelastic deformations take place in the form of localised permanent yielding at joints between girders and columns, which dissipates energy induced in the building frame by ground motion. It is obvious that adequate ductility capacity is crucial if a building frame is to be designed for

earthquake induced forces lower than those that will result if the frame remained elastic.

Ductility capacity varies with the type of construction and at present, reliable estimates of this capacity can only be obtained experimentally. The materials for construction commonly used in the construction of buildings located in seismic areas are steel, concrete or a combination of the two. Information should therefore be obtained on the ductility capacity of members made from these materials.

A fair number of experiments have been carried out in the past to obtain cyclic load-deformation characteristic of homogeneous steel and reinforced concrete beams strained into the inelastic range. To the best knowledge of the author, no experiment of similar nature has been performed on composite structures. It is the purpose of this thesis to examine by experiments, the cyclic load-deformation characteristic (i.e. Hysteresis loops) of steel and concrete composite beam-to-column joint stressed into the inelastic range in an attempt to appraise the ductility capacity of this commonly used type of construction under cyclic loading.

1.2 REVIEW OF PREVIOUS RESEARCH

Previous research on ductility capacity of beam-to-column joint has been concentrated in two areas: steel, and

reinforced concrete. The research on cyclic load behavior of homogeneous steel subassemblages and reinforced concrete members has been fairly extensive, but little work has been done on steel-concrete composite members. Experiments on composite sections have been directed towards the determination of positive moment-rotation characteristic under monotonic loading. A few experiments have been carried out to investigate the negative moment capacities of composite steel-concrete beams under statically applied monotonic load. However, the previous research in the above areas provides useful information for the present field of study, and has been of great assistance to the work presented in this thesis. The more relevant of this previous work is briefly reviewed below.

In 1972, V.V. Bertero, E.P. Popov and Helmut Krawinklar [3] tested four subassemblages of steel beam-to-column joints of a multistory unbraced frame as shown in Figure 1.2, under closely simulated earthquake forces as shown in Figure 1.3. The results of this investigation showed that steel beam-to-column joints had high degree of energy absorption and dissipation. Failure was generally due to local instability in flanges in the plastic region of the beam, and also due to large shear deformation in panel zones.

In 1973, Egor P. Popov, and Vitelmo V. Bertero [4], both of University of California, Berkeley, investigated the

ductility property of the various kinds of steel beam-to-column connections as shown in Figure 1.4. The loads were applied in a quasi-static manner as shown in the cyclic load sequence diagram in Figure 1.5a and load deflection relationships were obtained as shown in Figure 1.5b. It was concluded that due to strain hardening effect, all types of connections tested developed strengths in excess of that predicted on the basis of simple plastic theory. Hysteresis loops obtained from those tests were remarkably stable, and a skew-symmetric bilinear moment-curvature skeleton curve was recommended for seismic analysis.

Response of simply supported reinforced concrete beams under cyclic loading was studied by G.L. Agrawal, Leonard G. Tulin and Kurt H. Gerstle of Colorado University [5] in 1965. Theoretical prediction was compared with experimental observations. It was concluded that sufficiently accurate moment-curvature response of doubly reinforced beams can be obtained by analytical method, that the response was predominantly affected by the steel, and that there was only a slight influence of the concrete part of the beam.

In 1972, Robert Park, Dudley C. Kent and Richard A. Sampson [6] tested a series of nine reinforced concrete beam-to-column joints under cyclic loading. Figure 1.6a shows typical dimensions of the specimens. Theoretical moment-curvature

curves were derived and compared with experimental results as shown in Figure 1.6b. It was observed that the theoretical curves compared reasonably well with experiment, and that cracking in concrete was a major factor in the degrading stiffness response of concrete structure under cyclic loading. An idealised degrading stiffness moment-curvature response curve was suggested for dynamic analysis of beams.

In a comprehensive report prepared for the U.S. National Science Foundation in January, 1973, Mehmet Celebi and Joseph Penzien of California University [7] investigated the influences of stirrups spacing and ~~shear~~ span to depth ratios upon energy absorption, stiffness deterioration, strength and deflection ductility of reinforced concrete beam-to-column joints. Both quasi-static and dynamic loading tests were carried out. It was observed that for reinforced concrete structures, the non-linear behavior of force-deformation hysteresis loops is influenced by shear deformation to a significant extent. The influence of dynamic loading on stiffness degrading and energy absorption properties was not significant, except that yield strength of the tested specimens appeared to have increased by as much as 20%.

In 1968, Piepgrass of University of Alberta [10] tested six composite beams with different slab width and reinforcement, to study the behavior of composite beams in isolated

negative moment region. It was found that the ratio of moment at failure to the computed plastic moment capacity of the section ranged from 0.98 to 1.07. It was also concluded that a steel section classified as compact by CSA Standard S16 does not necessarily possess adequate rotation capacity when used in a composite beam.

In 1969, J.H. Davison and J. Longworth [11] of the University of Alberta, tested seven steel-concrete composite beams with different section properties as indicated in Figure 1.7 under monotonically applied static load causing negative moment. A steel wide flange section, W12 x 36 was used as the main beam throughout. It was concluded that:-

- (1) Significant increases in the negative moment capacity of composite beams can be achieved by the addition of longitudinal slab reinforcement, but these increases are not in proportion to theoretical simple plastic moment values.
- (2) Rotation capacity of negative plastic hinge decreases with increasing amount of longitudinal slab reinforcement in the slab.
- (3) If longitudinal reinforcement remain unchanged, variation in slab width within the limits permitted by CSA Standard S16, has no significant effect on ultimate moment capacity or rotation capacity.

It was recommended that the full contribution of longitudinal slab reinforcement not be used in calculation of the simple plastic moment capacity of composite beams in negative bending, and also that effective slab widths for negative bending be the same as those specified by CSA S16 for positive bending.

In 1970, J.H. Daniels, G.D. Kroll and J.W. Fisher [12] tested two full size model of composite beam-to-column joint. Figure 1.8 shows the two specimens (J1 & J2) tested in the investigation. The specimens were loaded as shown in Figure 1.9, to simulate the combined moment effect of gravity and wind load at various region along the continuous span of a typical floor of a multistory structure. It was concluded that:-

- (1) At a joint where negative moment existed on one side of the column and positive moment existed on the other side, slab width for calculating flexural capacity of the composite section in the positive region can be taken as the width of the column face.
- (2) Considerable ductility existed at the joint to enable plastic design theory to be applicable for analysis.

(3) Ultimate strength theory gave accurate prediction of ultimate flexural capacity for the joint.

(4) For discontinuous composite beams, no composite flexural strength existed in negative moment region.

(5) For continuous composite beams, local buckling of the steel beam in negative moment region must be prevented if ultimate strength of the composite beam is to be reached.

In 1972, J. Jay Climenhaga and R. Paul Johnson [13] investigated the effect of local flange and web buckling on the rotation capacity of composite beam-to-column joints. Typical elevations of test specimens and their properties are shown in Figure 1.10 and Figure 1.11 respectively. Static three-point load to produce negative bending moment was used in all tests. The specimens were divided into two categories: unstiffened and stiffened (longitudinally on both sides of web), and their moment rotation characteristics were compared.

It was concluded that the then British slenderness rules ensured the attainment of simple plastic moment in unstiffened section, but did not adequately provide for rotation capacity. A set of new equations were suggested for limiting the slenderness ratios of unstiffened steel sections used in composite construction. For longitudinally stiffened sections,

it was found that web deformation could be prevented by suitably selected longitudinal web stiffeners in accordance with the recommendations outlined in the study. These recommendations suggested that length of such stiffeners to be about 1.5 times the depth of the steel section and their cross-sections be chosen to have at least the same torsional stiffness as a flange.

In 1972, J.W. Fisher, J. Hartley Daniels, and Roger G. Slutter [14] of Fritz Engineering Laboratory, Lehigh University, studied the various parameters influencing the fatigue strength of continuous composite beams, such as the influence of shear connectors, slab cracking, optimum reinforcement, effective width, creep and shrinkage in negative moment region. Two types of test beams were used for the investigation as shown in Figure 1.12 - continuous test beams and negative moment test beams. Cyclic load of 2,000,000 cycles was applied to the continuous type and static load to the negative type. It was concluded that crack width in negative moment region decreases with an increase in both the reinforcement and perimeter ratio and it was recommended the minimum area of longitudinal reinforcement in negative moment region for crack control be one percent of the cross-sectional area of the concrete slab.

In 1972, R.P. Johnson of University of Warwick and M. Hope-Gill of University of Cambridge [15], proposed the concept of 'Semi-rigid joint' design for composite structures, by using moment resisting longitudinal steel reinforcement in concrete slab at negative moment region as shown in Figure 1.13. The investigation was intended to introduce and test the Semi-rigid joint concept as a compromise solution between the 'Simple joint' which neglect negative moment of resistance and the 'Rigid joint' which was designed to have negative moment all resisted by the connection of steel beam alone. Five specimens were tested; results showed that 'Semi-rigid' joints provided not only predictable negative flexural strength and large rotational capacity, but also improved the crack control of concrete slab.

A cyclic load experiment was performed on encased steel and concrete beam-to-column assembly, by Kiyoshi Muto of Tokyo University [16]. Details of a typical specimen are shown in Figure 1.14. It was observed from the obtained test results that the encased structure possessed high ductility capacity.

1.3 SCOPE OF THE PRESENT STUDY

The objective of the present study was to obtain an estimate of the ductility capacity of a beam-column joint in

which the beam had a composite steel concrete section and the column was of steel. To achieve this aim, cyclic load deformation characteristics of typical beam-column joints were obtained through experiments on three specimens. The first of these was a plain steel specimen. The other two had steel column stubs and steel-concrete composite beams. This report presents a detailed description of the experiments performed and the results obtained. The report has been organised into five chapters and the contents of each chapter are briefly described below.

Chapter I presents an introduction and a review of the existing literature relevant to the present study.

The details of the test specimen, the testing machine and the instrumentation is given in Chapter II.

Chapter III presents the experimental results obtained from the tests. Hysteresis loops are plotted between moment and curvature, moment and rotation, and load and deflection. The variations of strains in longitudinal reinforcement along the width of slab are also plotted.

Chapter IV discusses the test results obtained and compares the measured moment capacity with those predicted on the basis of simple plastic theory.

The conclusions drawn from the test results and a summary of the work done are presented in Chapter V together with suggestions concerning further scope of research in this field of study.

----- oooOooo -----

FIGURE 1.1

FIGURE 1.2

Specimen B-2

Specimen B-1

FIGURE 1.3

Specimen number (1)	Size (2)	Description of connection (3)
1	W 18 x 50	Five 7/8 in. bolts
2		All welded
3		Five 3/4 in. bolts
4		Four 3/4 in. bolts
8	W 24 x 76	No web connection
5		Seven 7/8 in. bolts
6		Seven 1 in. bolts
7		All welded

SECTION A-A

SECTION B-B

FIGURE 1.4

FIGURE 1.5a

FIGURE 1.5b

FIGURE 1.6a

FIGURE 1.6b

BEAM	SLAB WIDTH	LONGITUDINAL REINFORCEMENT	A_s (sq.in.)	A_g/A_{WF}	A_s/A_c	TRANSVERSE REINFORCEMENT	A_t/A_c
11	4'-0"	6 #4 BARS	1.18	.111	.00615	#3 BARS @ 6"	.00468
12	4'-0"	10 #4 BARS	1.96	.185	.00995	#3 BARS @ 6"	.00468
13	4'-0"	8 #5 BARS	2.45	.232	.0128	#3 BARS @ 4½"	.00605
14	4'-0"	12 #5 BARS	3.68	.348	.0192	#3 BARS @ 4½"	.00605
15	6'-0"	12 #5 BARS	3.68	.348	.0128	#3 BARS @ 4½"	.00605
16	2'-8"	8 #5 BARS	2.45	.232	.0192	#3 BARS @ 4½"	.00605
17	6'-0"	8 #5 BARS	2.45	.232	.00850	#3 BARS @ 4½"	.00605

FIGURE 1.7

FIGURE 1.8

SCHEMATIC ELEVATION OF J1 AND J2

BENDING MOMENT IN UNBRACED MULTISTORY FRAMES

FIGURE 1.9

TYPICAL ELEVATIONS OF TEST SPECIMENS
WITH A STEEL FLAT

TYPICAL ELEVATIONS OF TEST SPECIMENS
WITH A CONCRETE SLAB

FIGURE 1.10

Group	Specimen	Joist Size	b/t	d/w
1	SB1	5 x 3 RSJ	9.9	23
	SB7	5 x 3 RSJ	9.9	23
2	SB3	8 x 5½ UB20	14.0	31
	SB9	8 x 5½ UB20	14.0	31
	HB40	8 x 5½ UB20	14.0	31
3	SB2	8 x 5½ UB17	16.5	33
	SB8	8 x 5½ UB17	16.5	33
	HB41	10 x 4 UB15	15.1	40
	SB13s	8 x 5½ UB17	16.9	33
4	SB14	12 x 4 UB22	10.0	46
	SB4	12 x 4 UB16-5	14.1	50
	SB10	12 x 4 UB16-5	14.1	50
	HB42s	12 x 4 UB16-5	14.9	48
5	SB5	14 x 5 UB22	14.9	56
	SB11	14 x 5 UB22	14.9	56
6	SB6	16 x 5½ UB26	16.7	62
	SB15s	16 x 5½ UB26	16.2	61

FIGURE 1.11

(a) Continuous Test Beam.

(b) Negative Moment Test Beam.

FIGURE 1.12

SEMI-RIGID JOINT.

FIGURE 1.13

FIGURE 1.14

CHAPTER II
TEST PROGRAM

2.1 TEST SPECIMENS

2.1.1 CHOICE OF GENERAL DIMENSIONS

Three specimens were built and tested. In the first test, which was designed to be a pilot test, a plain steel subassemblage (SB1) was used. Steel-concrete composite subassemblages (SB2 and SB3) were used in the second and third tests.

Based on a review of the previous research, it was decided that a near full-scale specimen simulating a beam-to-column subassemblage of a common multistory building frame would be appropriate. The specimen size was selected to represent a twenty-foot span between interior columns and ten-foot spacing between floor beams which is a common arrangement in multistory buildings. It was assumed that the points of counter-flexure are located in the beam at quarter span, that is, at five feet on both sides of the column stub. An effective span of ten feet was therefore selected for all the three specimens. For specimens SB2 and SB3, the width of concrete slab was chosen to be five feet. This conformed to CSA Standard S16, Clause 21.3.1 which limits the effective slab width to $\frac{1}{4}$ of the span. The slab thickness was chosen to be 4 inches. Figures 2.1 through 2.6 show the details of the three specimens.

2.1.2 CHOICE OF STEEL BEAM SECTIONS

From a review of the previous research, it was evident that in order to effectively study the rotation capacity of the composite beam-to-column subassembly, premature local and lateral buckling of the compression elements of the steel section had to be prevented by selecting a suitable section and employing suitable lateral bracing. Since the concrete slab could be considered to provide effective lateral bracing, the criterion for the selection of a suitable steel section became one of resistance to local buckling of webs and flanges.

The physical behavior of a composite beam with longitudinal slab reinforcement when subjected to negative moment is similar to that of a beam-column. The tension force in reinforcement acts as a longitudinal load on the steel section. This longitudinal load is transmitted to the steel section through shear connectors. CSA Standard S16 limits the depth-thickness ratio of webs of compact sections when subjected to axial compression plus bending to $255/\sqrt{F_y}$. Similarly, to preclude the possibility of local buckling until the onset of strain-hardening, CSA Standard S16 limits the width-thickness ratio of the projecting elements of the compression flange to $54/\sqrt{F_y}$. For CSA G40.21 44W steel with a yield stress of 44 ksi these ratios are 38.4 and 8.13 respectively.

In view of these limitations, a W12x27 was selected as the steel section to be used for all the three tests. For this section the h/w and b/t ratios for the web and flange are 42.72 and 8.12 respectively. The h/w ratio was thus slightly in excess of the allowable.

2.1.3 CHOICE OF A STEEL COLUMN SECTION

The column section was intended to be short, its size to be proportional to the selected W12x27 beam. The column section was not intended to resist transverse load under the loading arrangement of the tests. A W10x33 was considered adequate and was chosen to be the column stub for all the three specimens.

2.1.4 PROPORTIONING OF COMPOSITE SECTIONS

With the selected general dimensions and size of steel beams as described in the previous sections, the remaining constraints for designing the composite section were the loading capacity and displacement range of the testing machine. The maximum load capacity of the machine was 180 kips tension or compression, and the total stroke of the piston was 5 inches giving a displacement range of ± 2.5 inches.

Using the plastic method of analysis, both SB2 and SB3 specimens were designed to have a positive plastic moment of 257.6 kips-ft., which was 1.85 times the plastic moment of

resistance of the steel beam alone. The load required from the machine to produce this moment was 103 kips, leaving sufficient margin for excess strength on account of possible higher strength of concrete and strain hardening of steel.

In practice, negative moment capacity of a composite beam can be designed to lie in the range of between 0.5 and 0.85 times its positive plastic moment capacity. Specimen SB2 was designed for a negative plastic moment capacity of 75% of the positive plastic moment capacity while SB3 was designed to have a negative moment capacity equal to 85% of the positive value. Detail design calculations are shown in Appendix A.

2.1.5 SLAB REINFORCEMENT

The purpose of introducing slab reinforcement over support is to provide negative moment of resistance in the composite beam. Increasing amount of longitudinal slab reinforcement in a negative moment region will move the neutral axis from within the steel beam towards the slab. When neutral axis reaches the tension flange, additional reinforcement will only increase the plastic moment capacity of the section by a slight amount. For this reason, it is efficient to keep the position of the neutral axis at or below the tension flange. It was found by calculation that for a negative moment capacity of 0.75 or 0.85 of the

positive moment capacity of the specimen, the neutral axis lies within the top flange of the steel beam. These negative moment capacities were thus more than what will be aimed for in a practical design. The longitudinal reinforcement required for a negative moment capacity of 75% was 2.45 sq. in. (8 # 5 deformed bars) and that required for a moment capacity of 85% was 4.81 sq. in. (8 # 7 deformed bars).

In addition to the longitudinal reinforcement, transverse steel at the rate of .5% of the concrete cross-section (#3 bars @ 5.5 inches centres) was used to control longitudinal cracking due to shrinkage and temperature and to distribute the longitudinal strains throughout the width of the slab. Detail calculations are shown in Appendix A.

2.1.6 PANEL ZONE

Column web stiffeners were provided on the column web opposite to the flanges of the connecting beams, in order to prevent curling of column flanges under tension in the beam flanges, and buckling of the column web under compression in the beam flanges.

In accordance with the requirement of Clause 24.4 of CSA Standard S16, only 3/8" thick horizontal stiffeners were needed for the specimen. However, additional cross-braced stiffeners of 1/2" thick were used in all the three specimens for extra rigidity at panel zone and to prevent

buckling of column web at ultimate state.

2.1.7 SHEAR CONNECTORS

Sufficient shear connectors were provided to withstand the force in the longitudinal reinforcement at yield when the subassembly was subjected to negative moment and the total compressive force in the concrete slab when the specimen was subjected to positive moment. The connectors were spaced uniformly throughout the span. The capacity of 0.75 inch diameter 2.5 inches long headed stud shear connectors was taken as 23 kips for concrete with a compressive strength of 3000 psi as outlined in CSA Standard S16.

2.1.8 WELDING AT BEAM-TO-COLUMN JOINT

In order to preclude the possibility of premature failure at welds, Type 7028 weld electrode of high tensile strength (80 ksi) higher than of the structural steel (44 ksi) was used. The beam flanges were butt welded to the column-stub flanges. The beam web was welded to the column flange by means of a $\frac{1}{4}$ inch fillet weld. Details of the welds are shown in Figure 2.7.

2.2 TESTING APPARATUS

2.2.1 ROLLER SUPPORTS

The subassemblies were tested with the concrete slab over the steel beam. The steel beam was secured to roller

supports at both ends, so that the ends of the beam were free to move in the horizontal direction.

Details of the Roller Supports assembly are shown in Figure 2.8. Each assembly consisted of two eighteen-inch long W18x40 standing upright with one-inch end plates welded to the ends, and secured to the lower steel beam of the loading frame by high strength (A325) bolts. Steel blocks 3" x 6" x 18" bolted to the top of the W18x40 section provided bearing surfaces for the rollers. Two steel blocks each 5-1/16" thick by 4½" wide and 6" long resting on top of the 3" thick steel blocks acted as stoppers for the rollers and also provided support for two reaction beams of W6x15.5 section. These reaction beams were held down by eight ¾" diameter 18 inches long high strength bolts and provided the necessary resistance to the uplift force at supports. A five-inch diameter steel roller ran between the bearing blocks and the W6x15.5 reaction beams. The roller threaded through a 5" standard pipe which in turn was secured to the lower flange of the test specimen by a welded bracket.

Each support assembly was capable of providing 95 kips of holding down force and upward reaction. PLATES 2.1 and 2.2 are photographs of the support assemblies.

2.2.2 TESTING EQUIPMENT

An MTS closed-loop servo controlled, electro-hydraulic structural testing system was used for the test. The 185-kip capacity (tension/compression) system consisted of four main units:

- (1) Control Console Assembly
- (2) Hydraulic Actuator
- (3) Hydraulic Power Supply
- (4) Testing Frame

(1) Control Console Assembly was a combination of the following modules:-

- a. Control Panel - To provide centralized manual and automatic control for console power on/off, hydraulic pressure on/off, and the simultaneous starting and stopping of programmer and recorder.
- b. Operator's Control - To provide power and control for Servo Controller, DC Transducer Conditioner, AC Transducer Conditioner and Feedback Selector. Servo Controller is an all DC, solid state device providing a signal for servo-valve control depending on the difference between a program signal input and feedback signal. DC Transducer Conditioner provides DC excitation and output amplification for the strain gage bridge in the

load cell. Four distinct operating ranges are available: ± 200 kips, ± 100 kips, ± 50 kips, ± 20 kips. AC Transducer Conditioner provides 10 KHZ excitation for use with displacement transducer. Its output is 10 volts DC signal full scale for feedback input to the servo controller or for monitoring purposes. Feedback Selector allows switching of up to two feedback signals (load and stroke) to be applied to servo controller.

- c. Functions Generator - To provide six cyclic waveshapes plus an inversion capability for each waveshape. Frequency of the cyclic waveshapes is variable from 0.001 Hz to 1100 Hz. A ramp output is provided with a time base from 0.5ms to 500 seconds. The generator includes a single-cycle trigger, ramp hold and provisions for remote control. All waveforms start at zero and are selectable to proceed initially at either a positive (+) or a negative (-) sense.

(2) Hydraulic Actuator was normally rated at 185 kips static with a 62.7 square-inch piston area, equipped with an aluminum manifold having a 40 gpm servo-valve attached. The actuator assembly also included an integrally mounted, 5-inch LVDT (Linear Variable Displacement Transformer) type of displacement

transducer. The maximum stroke of the piston was 5 inches.

(3) Hydraulic Power Supply unit was a constant volume, 3000 psi and 30 gpm flow capacity system equipped with a 40 hp, 460 volts, 3-phase and 60 Hz motor.

(4) Testing Frame was a 14-foot tall, 9-foot wide structural steel frame in which the hydraulic actuator was mounted. Figures 2.9 (a) and (b) show details of the testing frame.

A summary of the overall specifications and performance of MTS system is listed in Appendix B. Photographs of the set up of the three tests are shown in PLATES 2.3 through 2.8.

2.3 INSTRUMENTATION

2.3.1 STRAIN GAGES

M-M (Micro-Measurements) type CEA-06-500UW-120 electrical resistance one-arm strain gages having the following specifications were used to measure strains in the slab reinforcement and in the steel sections:

Strain Limit : 30,000 to 50,000 microstrain (3 to 5%),
tension or compression.

Resistance : $120.0 \pm 0.3\%$ ohms.

Fatigue Life : Over 10,000,000. cycles at ± 1400 micro-strain; over 1,000,000. cycles at ± 1500 microstrain or approximately 2800 micro-strain unidirectional (tension or compression).

Twenty strain gages were installed on test specimen SB1, eight on the web and two on the flanges of the steel beam on both sides of the column stub as shown in Figure 2.10. A total of thirty-six strain gages were used for specimen SB2. Sixteen of these were located on the web of the W12x27 steel section and the rest were installed on the longitudinal reinforcement of the concrete slab and arranged as shown in Figure 2.11. The locations of strain gages for specimen SB3 are shown in Figure 2.12; a total of thirty-two strain gages were used, sixteen on the web of the steel beam and sixteen on the longitudinal reinforcement of the slab.

All strain gages were connected to a B & F model SY161 multi-channel digital strain indicator. The indicator was capable of automatic scanning of the strain gages at the rate of two gages per second, processing of the signal, and direct printing of strain on a paper tape. The lowest strain that could be recorded was 10^{-4} . A portable strain indicator accompanied by a ten-channel switch-and-balance unit was used as a back-up system to the B & F indicator. Three-wire system of strain gage connection was used for the purpose of temperature compensation.

Strain gages mounted on a vertical line on the steel section made it possible to measure the average curvature at the section containing the line.

2.3.2 MECHANICAL DIAL INDICATORS

'Starett' type model 25-441 and 25-5041 dial indicators were used for measuring linear displacements. Model 25-441 had a range of one inch, while model 25-5041 had a range of five inches. Two model 25-441 indicators were arranged at each end of the specimen as shown in Figure 2.13 for measuring end rotations. The 5-inch range indicator shown in Figure 2.14 was used for measuring the central deflections of the subassemblage. Layout of the dial indicators is shown in PLATES 2.9 and 2.10.

2.3.3 X-Y RECORDER

In all three tests, an X-Y recorder was used to obtain approximate load-deflection hysteresis loop instantly as the tests proceeded. The recorder was connected to the transducer conditioner modules of the MTS testing system. It plotted the central deflection on the Y-axis through a signal from the displacement transducer mounted in the hydraulic actuator, and the load on the X-axis through a signal from the load cell which was also mounted in the hydraulic actuator of the MTS system.

2.4 MATERIAL PROPERTIES

2.4.1 CONCRETE

Ready-mixed concrete made from CSA Normal Portland Cement conformed to CSA Standard A5 was used for casting the concrete slabs of specimens SB2 and SB3. Three concrete cylinders were tested for each of SB2 and SB3 on the day of the experiment.

The average tested compressive strength of concrete for specimen SB2 was found to be 3926 psi, and that for specimen SB3 was 4134 psi. Figure 2.15 and 2.16 show the stress-strain curves obtained from the cylinder tests.

2.4.2 STRUCTURAL STEEL

In all three specimens, the beam was of hot-rolled CSA, G40.21, 44W steel which had a specified yield strength of 44 ksi. Specimens SB1 and SB2 were made from steel of the same heat lot. Two tensile test coupons were taken from the web of an extra length of the steel section and tested to obtain the stress-strain curve under tension.

Yield stress of the steel of specimens SB1 and SB2 was found to be 43.22 ksi, and that of specimen SB3 was 56.24 ksi. Stress-strain curves of the tensile tests are shown in Figures 2.17 and 2.18.

2.4.3 STEEL REINFORCEMENT

Deformed bars of specified yield stress 60 ksi were used for the slab reinforcements of specimens SB2 and SB3. Two coupons were tested to determine the yield strength of the deformed bars. The results showed that the bars used in SB2 had a yield stress of 52.16 ksi, and those used in SB3 had a yield stress of 62.23 ksi.

2.5 FABRICATION OF TEST SPECIMENS

Steel beams for specimens SB1 and SB2 were fabricated from G40.21, 44W material obtained from the same lot. Specimen SB3 was fabricated from the same material but at a later date.

For specimen SB1, fabrication involved the installation of cross bracing stiffeners, end bearing plate at top of column stub, bearing stiffeners and bearing pipes at both supports by means of electric welding. Electrical resistance strain gages were attached to the steel section after fabrication.

The fabrication of SB2 and SB3 was divided into two phases. The first phase was the same as that of specimen SB1, but with the additional process of welding two rows of shear connectors onto the top flanges of the steel beams. Also, in specimen SB3, horizontal stiffeners were welded to

the web of the steel beams. The second phase was the casting of the concrete slab, which involved construction of timber formwork, installation of electrical resistance strain gages on the surface of the longitudinal reinforcements, placing the reinforcement in the formwork according to the design and finally casting and curing of concrete.

Two different methods were employed to protect the electrical strain gages attached to the reinforcement from possible damage caused by concrete. For specimen SB2, fibre glass tape was wrapped round the reinforcing bar in several layers at the locations of the attached strain gages. A thin film of oil was applied onto the surface of the strain gages before the use of tape to allow free movement between the gages and the adhesive surface of the protective fibre glass tape.

Another method of strain gage protection was used in specimen SB3. A thin metal hollow section of size 1.5" x 0.5" was cemented onto the reinforcing bar by 'EPOXY' glue at each location of the attached strain gages. This way the strain gages were prevented from coming into contact with concrete. PLATES 2.11 and 2.12 show the pictures of the two methods of strain gage protection.

All specimens were white-washed with lime a day before testing. The purpose of the white paint was to

display yield line patterns through flaking of the paint on the yielded portion of the specimens.

2.6 TEST PROCEDURE

2.6.1 PRE-TEST PREPARATIONS

The hydraulic power supply unit of the MTS testing system and the B & F strain indicator were allowed to warm up for an hour. The loading piston of the hydraulic actuator was brought down from its suspended position onto the top bearing plate of the column stub. The top bearing plate of the column stub was then connected to the piston swivel head by four one-inch diameter A325 bolts.

The tightening of the bolts induced a small upward force on the specimen, partly off-setting its self weight. At this stage the load display on the MTS display was adjusted to zero. A zero adjustment was also carried out on the displacement scale.

Each electrical resistance strain gage circuit was balanced to ensure an initial reading of zero after the loading piston had been secured. In this way, the initial strain caused by the bolting process was eliminated from the reading.

Load deflection scale on the X-Y recorder was calibrated to a suitable range. Mechanical dial indicators were set at the centre of their operating range to enable linear displacement measurements in two directions.

2.6.2 FIRST EXPERIMENT - (SPECIMEN SB1)

A concentrated cyclic force was applied to the specimen through the column stub in a quasi-static loading sequence shown in Figure 2.19. The selection of magnitude and number of load reversals was arbitrary.

Basically, the specimen was first subjected to three complete cycles of displacements to produce stresses corresponding to those produced by the service load capacity of the specimen (i.e. below yield stress). The amount of central displacement was then increased to a value approximately equal to the yield deflection of the specimen, and this displacement was repeated for three complete cycles. The maximum loads in all subsequent load cycles produced stresses in the non-elastic range.

The points along the load cycles where readings were taken are shown in Figure 2.20. Central deflection, end rotations, applied central load and strain measurements were all recorded simultaneously at each of the reading points while the central deflection was held constant.

The recording procedure was repeated as the test progressed. The specimen failed through a fracture of the butt weld between the top flange of the beam and the column stub.

2.6.3 SECOND EXPERIMENT - (SPECIMEN SB2)

A concentrated cyclic force was applied to the specimen through the column stub in a quasi-static loading sequence as shown in Figure 2.21. The selection of magnitude and number of load reversals was arbitrary.

The specimen was first subjected to two complete cycles of displacements to produce stresses corresponding to the service load capacity of the specimen (i.e. below yield stress). During the third and fourth load cycles, maximum deflections were increased to approximate the yield deflections of the specimen in both the positive and negative directions.

Because the section was much stiffer under downward load than under an upward load, the applied downward deflections were kept at between 0.05" to 0.10" less than the upward deflections for the first eight cycles of loading. Maximum displacements in subsequent load cycles were then increased approximately by 0.05" to 0.10" each cycle in the upward direction, but were increased only slightly beyond

0.30" in the downward direction.

The points along the path of the load cycles where readings were taken are shown in Figure 2.22. Central deflection, end rotations, applied central load and strain measurements were all recorded simultaneously at each of the reading points when applied load was allowed to stabilize and held constant.

The recording procedure was repeated as the test progressed until the specimen failed under negative loading by buckling of flanges and web of the steel beam and the load carrying capacity of the specimen declined.

2.6.4 THIRD EXPERIMENT - (SPECIMEN SB3)

A concentrated cyclic force was applied to the specimen through the column stub in a quasi-static load sequence shown in Figure 2.23. The selection of magnitude and number of load reversals was arbitrary.

After subjecting the specimen to initial cycles of loading within the elastic range, the specimen was subjected to two complete cycles of loading which produced central deflections equal to the yield deflections of the specimen both in the upward and the downward directions.

Because the specimen was much stiffer under downward loading than under upward loading, the ratio of downward

deflection to upward deflection in the third cycle through the fifth cycle was kept approximately at two-third. The maximum downward loads in subsequent cycles were then increased in steps of about 5 kips every cycle while the incremental upward deflection per cycle was set approximately at 0.1".

The points along the path of the load cycles where readings were taken are shown in Figure 2.24. Central deflection, end rotations, applied central load and strain measurements were all recorded simultaneously at each of the reading points when the applied load was allowed to stabilize and held constant.

The recording procedure was repeated as the test progressed. When sufficient number of cycles of loading had taken place the displacement was increased in the upward direction till failure took place by buckling of the beam flange and web.

APPLIED LOAD

FIGURE 2.1 SPECIMEN No. SBI

FIGURE 2.2 SPECIMEN No. SB2

FIGURE 2.3 SPECIMEN No. SB3

Reinforcement Layout for Specimen SB2.

FIGURE 2.4 -I

Reinforcement Layout for Specimen SB3

FIGURE 2.4 -II

SPECIMEN SB2

SPECIMEN SB3

FIGURE 2.5 TYPICAL CROSS SECTIONS

FIGURE 2.7 TYPICAL WELDED CONNECTION FOR SPECIMENS SB1, SB2 & SB3.

FIGURE 2.8 ROLLER SUPPORT UNIT.

FIGURE 2.9 a FRONT VIEW OF TESTING FRAME

FIGURE 2.9b - SIDE VIEW OF TESTING FRAME

FIGURE 2.10 - LOCATIONS OF STRAIN GAGES FOR SPECIMEN SB1.

FIGURE 2.11 LOCATION OF STRAIN GAGES FOR SPECIMEN SB2.

FIGURE 2.12 - LOCATION OF STRAIN GAGES FOR SPECIMEN SB3.

FIGURE 2.13

FIGURE 2.14

FIGURE 2.15

FIGURE 2.16

FIGURE 2.17

FIGURE 2.18

FIGURE 2.19

FIGURE 2.20 - READING POINTS OF TEST No. 1

SEQUENCE OF DISPLACEMENT CYCLES FOR TEST No.2

FIGURE 2.21

FIGURE 2.22 - READING POINTS OF TEST NO. 2

FIGURE 2.24 - READING POINTS OF TEST NO. 3

Roller Support

Roller Support

Specimen SB1

PLATE 2.3

Specimen SB1

Specimen SB2

PLATE 2.5

Specimen SB2

Specimen SB3

Specimen SB3

Mechanical dial gages for the
measurement of end-rotation

**Mechanical dial gage for the
measurement of central deflection**

22

OF/DE

SPECIMEN SB2

SPECIMEN SB3

CHAPTER III

TEST RESULTS

3.1 INTRODUCTION

The original data obtained from all the three experiments are filed with the Department of Civil Engineering, Carleton University. Test data presented herein are shown in the form of graphs and tables.

3.2 GENERAL BEHAVIOR

3.2.1 SPECIMEN SB1

At low stress level, the vertical midspan deflection, curvature and end rotations were directly proportional to the applied load. However, these measurements showed a more rapid increase as the midspan moment approached the plastic moment of the beam. The concentration of additional rotation in the vicinity of the joint between the beam and the column stub in higher range of loading cycles was reflected by comparing the curvature obtained near the joint and the rotation measured at both ends of the specimen.

The specimen continued to take additional load beyond its simple plastic load with relatively large displacements. Local buckling of the flange started to take place near the beam column joint. However, the specimen continued to take additional load. Finally, failure occurred due to fracture

of the butt weld between the column flange and the top flange of the beam. Photographs of the specimen taken after the test showing yield patterns are shown in PLATE 3.1.

3.2.2 SPECIMEN SB2

Vertical midspan deflection, curvature and end rotations of the specimen remained linear under low intensity of loading cycles. Downward displacement was less than upward deflection under the same magnitude of load due to greater bending stiffness in the downward direction. In subsequent loading cycles, upward deflection, curvature and end rotations increased more rapidly as the midspan moment approached the plastic moment value. Fine cracks appeared on the surface of the concrete slab and yielding took place in the web and flanges of the steel beam.

Due to high moment gradient under high intensity load cycles, additional rotation of the specimen became concentrated in the vicinity of the joint between the steel beam and the column stub. The specimen continued to take additional load in both upward and downward directions. Finally, under upward load, local buckling first developed in the compression flanges of the steel beam adjacent to the column stub, followed shortly by buckling of the compression portion of the web. The test was terminated at this stage when the specimen continued to deflect under

decreasing loading. PLATE 3.2 shows the deformed shape of the specimen after failure.

3.2.3 SPECIMEN SB3

The behavior of this specimen was identical to that of Specimen SB2. Under increasing upward load, the compression flange buckled first, followed by the web. However, because of the longitudinal web stiffeners, the web buckling was delayed considerably. Eventually the web first buckled around the longitudinal stiffener and then the web and the stiffener buckled together laterally. The specimen stopped carrying any further load at this stage. PLATE 3.3 shows the deformed pattern of the specimen after failure.

3.3 SPECIMEN SB1

3.3.1 MOMENT-CURVATURE RELATIONSHIPS

Moment-curvature hysteresis loops are presented in Figure 3.1 in the form of parameter M_1/M_p plotted against curvature. M_1 is the applied bending moment at the point where the curvature was measured and M_p is the theoretical plastic moment of the beam obtained from the yield stress actually measured in tensile tests on coupons obtained from the beam.

The curve is plotted from the 49th reading through the 106th reading taken during the experiment. Moment-curvature

relationship was linear upto the 49th reading. Each curvature value represents the average of the curvatures obtained from two pairs of electrical resistance strain gages all mounted on the same vertical centre line on one side of the web of the beam.

3.3.2 MOMENT-ROTATION RELATIONSHIPS

Moment-rotation hysteresis loops are presented in Figure 3.2 in the form of parameter M_2/M_p plotted against θ/θ_p . M_2 is the applied bending moment at the point where the beam and the column stub meet, and M_p is the theoretical plastic moment capacity of the beam obtained from actual value of its yield stress.

The angle θ represents the total rotation between the two ends of the beam. The rotation at each end was obtained from dial gages bearing against a straight edge glued to the end of the beam. The gages were positioned 10 inches apart and the difference between their readings divided by the distance between the two gave the value of end rotation. Angle θ was then obtained by summing the two end rotations. Angle θ_p is defined by $\theta_p = \frac{M_p L}{EI}$, where L is the half span from the face of the stub to the centre of bearing, and I is the moment of inertia of the section. Figure 3.2 presents the data obtained from 49th reading through the 106th reading. The moment rotation curve was linear upto the 49th reading.

3.3.3 LOAD-DEFLECTION RELATIONSHIPS

Load-deflection hysteresis loops are presented in Figure 3.3 in the form of applied load plotted against central deflection. The plots represent the data obtained from the 45th reading through the 104th reading. The central deflection was measured by a mechanical dial gage with 5" stroke erected under the column stub. Due to inevitable tolerances in the bearing, a small amount of deflection due to slip was recorded as the load passed through zero value. A correction was made in the data to account for this slip. The plot of Figure 3.3 is based on this corrected data.

3.4 SPECIMEN SB2

3.4.1 MOMENT-CURVATURE RELATIONSHIPS

Moment-curvature hysteresis loops are presented in Figures 3.4a and 3.4b in the form of parameter M_1/M_p' plotted against curvature. M_1 is the applied bending moment at the point where the curvature was measured, and M_p' is the theoretical negative plastic moment of the composite beam obtained from values of the yield stresses actually measured in tensile tests on coupons obtained from the steel beam section and the longitudinal reinforcing steel.

Data obtained from the 27th reading through the 75th reading has been plotted on Figure 3.4a, while that obtained

from the 75th through the 139th reading has been plotted on Figure 3.4b. Each curvature value represents the average of the curvatures obtained from two pairs of electrical resistance strain gages mounted on one side of the web of the steel beam. The two gages of a pair were mounted on a vertical centre line on one side of the specimen centre, the other pair being mounted on a line symmetrically located on the other side of the specimen centre.

3.4.2 MOMENT-ROTATION RELATIONSHIPS

Moment-rotation hysteresis loops are presented in Figures 3.5a and 3.5b in the form of parameter M_2/M_p' plotted against θ/θ_p . M_2 is the applied bending moment at the point where the beam and the column stub meet, and M_p' is the theoretical negative plastic moment of the composite beam obtained from actual values of the yield stresses for the steel beam and the reinforcement.

Figure 3.5a presents the data obtained from the 27th reading through the 75th reading taken during the test, while Figure 3.5b presents the data obtained from the 75th reading through the 139th reading. Each value of angle θ represents the total rotation between the two ends of the beam obtained from the readings of four mechanical dial indicators, two at either end of the specimen. Angle θ_p is defined by $\theta_p = \frac{M_p' L}{EI'}$ where L is the half span from the face of the stub to the centre of end bearing and I' is the moment of inertia of the

composite section under negative bending.

3.4.3 LOAD-DEFLECTION RELATIONSHIPS

Load-deflection hysteresis loops are presented in Figures 3.6a and 3.6b in the form of applied load plotted against central deflection. Data obtained from the 27th reading through the 75th reading has been plotted on Figure 3.6a, while that obtained from the 75th reading through the 134th reading has been plotted on Figure 3.6b. Each value of central deflection represents the reading of a mechanical dial gage erected at midspan of the specimen. The dial gage readings were corrected for slip at the bearings.

3.5 SPECIMEN SB3

3.5.1 MOMENT-CURVATURE RELATIONSHIPS

Moment-curvature hysteresis loops are presented in Figures 3.7a and 3.7b in the form of parameter M_1/M_p' plotted against curvature in Figures 3.7a and 3.7b. M_1 is the applied bending moment at the point where the curvature was measured, and M_p' is the theoretical negative plastic moment of the composite beam obtained from actual values of the yield stresses for the steel beam and the reinforcement.

Data obtained from the 20th reading through the 66th reading has been plotted on Figure 3.7a, while that obtained from the 66th reading through the 153rd reading has been

plotted on Figure 3.7b. Each curvature value represents the average of the curvatures obtained from two pairs of electrical resistance strain gages mounted symmetrically with respect to the specimen centre as in the case of Specimen SB2.

3.5.2 MOMENT-ROTATION RELATIONSHIPS

Moment-rotation hysteresis loops are presented in Figures 3.8a and 3.8b in the form of parameter M_2/M_p' plotted against θ/θ_p . M_2 is the applied bending moment at the point where the beam and the column stub meet, and M_p' is the theoretical negative plastic moment of the composite beam obtained from actual value of the yield stresses in the steel beam and the reinforcement.

Figure 3.8a presents the data obtained from the 20th reading through the 66th reading, while Figure 3.8b presents the data obtained from the 66th through the 153rd reading. Each value of angle θ represents the total end rotation between the two ends of the beam while θ_p is defined as in the case of Specimen SB2.

3.5.3 LOAD-DEFLECTION RELATIONSHIPS

Load-deflection hysteresis loops are presented in Figures 3.9a and 3.9b in the form of applied load plotted against central deflection. Data obtained from the 20th reading through the 66th reading has been plotted on Figure

3.9a, while that obtained from the 66th reading through the 145th reading has been plotted on Figure 3.9b. Each value of central deflection represents the reading of a mechanical dial gage erected at midspan of the specimen. The dial gage readings were corrected for slip at the bearings.

3.6 LONGITUDINAL REINFORCEMENT STRAIN AT MIDSPAN REGION

Distribution of longitudinal reinforcement strain at midspan region across the concrete slab of specimens SB2 and SB3 are shown in Figures 3.10a and b and 3.11a and 3.11b respectively.

The strain values represent the average of strains in two bars placed symmetrically with respect to the longitudinal centre line of the slab, measured at the upward and downward load peaks.

3.7 CRACKING OF THE CONCRETE SLAB

The crack patterns of specimens SB2 and SB3 are similar as shown in PLATES 3.4 and 3.5. During early cycles of low load intensity, fine transverse cracks, starting from the corner of the column stub, occurred in the midspan region. As the loading cycles approached the negative yield load of the specimens, these cracks grew wider and additional transverse cracking occurred and spread towards the supports.

Diagonal cracks developed across the slab as ultimate upward load was approached.

3.8 ULTIMATE LOAD CONDITIONS

A summary of ultimate response of all the three specimens is shown in Table 3.1 in which ultimate moments and theoretical plastic moment capacities are compared in the non-dimensional form M_{ult}/M_p where M_{ult} is the measured ultimate negative moment capacity. Rotational capacity at ultimate moment θ_{ult} is also compared in Table 3.1, with angle θ_p as previously defined.

SPECIMEN	P _{ult} (kips)	M _{ult} (Kips-ft)	M _p ⁱ (Kips-ft)	M _{ult} /M _p ⁱ	θ _{ult} (× 10 ⁻⁴ rad.)	θ _p (× 10 ⁻⁴ rad.)	θ _{ult} /θ _p
SB1	+ 75.3	+172.51	+152.29	1.132	574	156	3.68
SB2	+ 90.0	-206.18	-207.50	0.993	402.50	134	3.00
SB3	+132.5	-303.35	-270.78	1.120	720.00	177	4.067

TABLE 3.1 - TEST RESULTS

D

FIGURE 3.1 EXPERIMENT NO. 1

EXPERIMENT NO. 1

FIGURE 3.2

EXPERIMENT NO. 1

FIGURE 3.3

FIGURE 3.10a

LONGITUDINAL REINFORCEMENT STRAIN
(INCHES PER INCH $\times 10^{-6}$)

(NOTE: Each point represents the average strain of two symmetrically placed reinforcing bars.)

FIGURE 3.10b

FIGURE 3.11a

LONGITUDINAL REINFORCEMENT STRAIN
(INCHES PER INCH $\times 10^{-6}$)

-600
-500
-400
-300
-200
-100
-0

EXPERIMENT No. 3

+278.11

+266.63

+226.34

+205.10

+127.14

+ 87.07

MOMENT (KIPS-FT.)

(Note: Each point represents the average strain of two symmetrically placed reinforcing bars.)

FIGURE 3.11b

SPECIMEN SB1

FRONT VIEW

REAR VIEW

SPECIMEN SB2

PLATE 3.2

SPECIMEN SB3

SPECIMEN SB2

SPECIMEN SB3

SPECIMEN SB3

CHAPTER IV

DISCUSSION OF TEST RESULTS

4.1 INTRODUCTION

In this Chapter, the test results reported in Chapter III will be examined to draw relevant conclusions. This will be done by referring to figures and tables already presented. Ductility capacity of the three specimens will be examined in terms of curvature, rotation and deflection. Energy absorption and ultimate strength properties will also be studied.

4.2 SPECIMEN SB1

4.2.1 MOMENT-CURVATURE RELATIONSHIPS

Moment-curvature curves for the specimen are shown in Figure 3.1. It can be seen that the hysteresis loops are very stable throughout the test. Yield started to take place around the 54th reading, and the hysteresis loop was reproduced during the next three equal cycles of loading ending at the 79th reading. At this stage the deflection amplitude was increased as indicated by the 80th reading. The unloading paths beyond the first yielding are parallel to the initial loading curve. Re-loading in an opposite direction showed an early yield as seen in Figure 3.1, indicating the Bauschinger effect.

The solid line in Figure 4.1 shows a typical moment-curvature relationship for a steel section. The relationship can be idealized by the curve 'oab'. Using the idealized curve

the curvature at yield can be defined as its value at point 'a' giving:

$$\phi_p = \frac{M_p}{EI} \dots\dots\dots(4.1)$$

where M_p is the plastic moment of the section. It is customary to use the above value of ϕ_p in defining the curvature ductility [2], which is given by:

$$\mu = \phi_{ult}/\phi_p$$

The plastic moment of the section was found to be 152.29 kips-ft. and E was 31520 ksi which give $\phi_p = 284.2 \times 10^{-6}$ per inch. The average of the curvature at the 93rd and 99th readings is 2314×10^{-6} per inch. The ratio ϕ_{ult}/ϕ_p known as the normalised hinge curvature capacity thus works out to 8.14 .

4.2.2 MOMENT-ROTATION RELATIONSHIPS

Moment-rotation hysteresis loops of the specimen are shown in Figure 3.2. The non-dimensionalised parameter Θ/Θ_p is the ratio of the total end rotations at any applied moment to the end rotation at the yield condition. Figure 4.1 shows a plot of moment, M/M_p versus rotation Θ , for a simply supported beam loaded at midspan. The change of slope at each support is denoted by $\Theta/2$. Θ_p denotes the total of the two end rotations that would correspond to a moment M_p at midspan, if the beam were to remain completely elastic. As such, Θ_p can be defined as:

$$\Theta_p = \frac{M_p l}{EI} \dots\dots\dots(4.2)$$

where M_p = negative plastic moment capacity of the specimen,

l = half length of the specimen from support to the face of the column stub.

The rotation ductility ratio is defined as Θ_{max}/Θ_p where Θ_{max} is the maximum end rotation attained by the specimen. The value of the ductility ratio is obtained from Figure 3.2 by averaging the Θ/Θ_p values of the 93rd and the 99th reading, and works out to 3.27 .

4.2.3 LOAD-DEFLECTION RELATIONSHIPS

Load-deflection hysteresis loops for the specimen are shown in Figure 3.3. The hysteresis loops are stable and very similar to the moment-rotation hysteresis loops shown in Figure 3.2. Maximum load attained in the test was 75.2 kips which is higher than the theoretical value of the true plastic load of 66.3 kips by 13.5%.

Figure 4.2 shows a simply supported beam with applied plastic load 'P' at midspan, that would correspond to a moment M_p at midspan if the beam was to remain completely elastic. As such, yield deflection Δ_y can be defined as:

$$\Delta_y = \frac{PL^3}{48EI} \dots\dots\dots (4.3)$$

where P = the theoretical value of the true plastic load,

L = the span length excluding the width of column stub,

E = modulus of elasticity,

I = moment of inertia.

The yield deflection of the specimen worked out to be 287.8×10^{-3} inch. From Figure 3.3, it is seen that the maximum upward deflection is 1365×10^{-3} inch at the 93rd reading.

The deflection ductility is then given by:

$$\frac{\Delta_{\max}}{\Delta_y} = \frac{1365}{287.8} = 4.74$$

The energy dissipated during a cycle of loading is measured by the area bounded by the hysteresis curve for that cycle. It is seen from Figure 3.1 that the area bounded by the outermost loop is large and symmetrical about the axis. $M_1/M_p = 0$, indicating that the specimen possessed equally high energy dissipation capacity in both positive and negative bending.

As stated earlier the failure of the specimen occurred by a fracture of the butt weld between the upper beam flange and the column stub. If the fracture had not occurred the specimen could probably take a little higher load, though not very much higher because flange buckling had already progressed. The ultimate moment at the face of the column stub was 172.9 kips-ft. The ratio of this moment to the true plastic moment capacity M_p , M_{ult}/M_p , works out to 1.132 as shown in table 3.1. The ultimate moment was thus about 13% higher than the true plastic moment, presumably due to the strain hardening of the steel.

4.3 SPECIMEN SB2

4.3.1 MOMENT-CURVATURE RELATIONSHIPS

The moment-curvature hysteresis loops are shown in Figures 3.4a and 3.4b. The downward deflections in the displacement cycles were maintained almost at a constant value after yielding while upward deflections were increased at a steady pace as indicated in Figure 2.21. As such the hysteresis loops obtained shows a gradual expansion toward negative moment and curvature.

As described earlier, the curvature at yield in upward deflection is defined as:

$$\phi_p = \frac{M_p'}{EI'} \dots\dots\dots (4.4)$$

in which M_p' is the true negative plastic moment of the specimen, and I' is the moment of inertia of the composite section under negative bending. The maximum curvature attained during the test (reading number 125) is 1963.28×10^{-6} per inch, while the yield curvature as worked out from equation 4.4 is 224.30×10^{-6} per inch. The normalised hinge curvature capacity ϕ_{ult}/ϕ_p is then given by:

$$\frac{\phi_{ult}}{\phi_p} = \frac{1963.28}{224.30} = 8.03$$

4.3.2 MOMENT-ROTATION RELATIONSHIPS

The moment-rotation response is shown in Figure 3.5a and

3.5b. Again, the non-dimensionalised parameter Θ/Θ_p indicates the ratio of the total end rotations at any applied moment to the end rotation at yield. As stated earlier, angle Θ_p is given by:

$$\Theta_p = \frac{M_p' l}{EI} \dots\dots\dots(4.5)$$

where M_p' = negative plastic moment capacity,

l = half length of the specimen from the support to the face of the column stub,

I' = moment of inertia of the composite section under negative bending.

From Figure 3.5b it can be seen that the ductility ratio Θ_{max}/Θ_p under negative bending moment is 3.00 (125th reading).

4.3.3 LOAD-DEFLECTION RELATIONSHIPS

Load-deflection hysteresis loops for the specimen are shown in Figures 3.6a and 3.6b. The maximum upward load attained in the test was 90 kips at the 125th reading, which is lower than the theoretical value of the true plastic load of 90.33 kips by 0.36%.

As stated earlier, upward yield deflection is given by:

$$\Delta_y = \frac{PL^3}{48EI'} \dots\dots\dots(4.6)$$

where P = the theoretical value of the true plastic load,

L = the span length excluding the width of the column stub,

E = modulus of elasticity,

I' = moment of inertia of the composite section under negative moment.

The yield deflection for the specimen worked out to be 247.5×10^{-3} inch. It is seen from Figure 3.6b, that the maximum upward deflection $\Delta_{\max}$ is 1011×10^{-3} inch at the 134th reading. The deflection ductility is then given by:

$$\frac{\Delta_{\max}}{\Delta_y} = \frac{1011}{247.5} = 4.08$$

The specimen demonstrated good energy dissipation in the negative moment response. Failure of the specimen took place at an ultimate negative moment of 206.18 kips-ft. at the face of the column. The true plastic moment capacity for this specimen was calculated as 207.50 kips-ft. as shown in Table 3.1. The ultimate moment therefore fell slightly short of the true plastic moment capacity. The specimen could not take any further load due to buckling of the flange and the web.

4.4 SPECIMEN SB3

4.4.1 MOMENT-CURVATURE RELATIONSHIPS

The moment-curvature hysteresis loops are shown in Figures 3.7a and 3.7b. In this experiment, the downward and the upward displacements were both increased proportionally at a constant pace as indicated in Figure 2.23. As such, the hysteresis loops obtained show a gradual expansion towards

both the positive and negative direction in moment-curvature regions.

From equation 4.4, the yield curvature of the specimen under negative moment worked out to be 322.2×10^{-6} per inch. The maximum curvature obtained during the test (reading number 141) is 1842.2×10^{-6} per inch, hence the normalized hinge curvature capacity is given by:

$$\frac{\phi_{ult}}{\phi_p} = \frac{1842.2}{322.3} = 5.71$$

The local buckling of web and flange was delayed till after the 150th reading of the test; much of this successful prevention of buckling was due to the horizontal stiffeners welded to the web of the beam. However, buckling first started in the flanges after the 150th reading and eventually the web buckled along with the stiffeners.

4.4.2 MOMENT-ROTATION RELATIONSHIPS

The moment-rotation response is shown in Figures 3.8a and 3.8b in terms of M_2/M_p versus θ/θ_p , where θ_p is defined in equation 4.5. From Figure 3.8b, it can be seen that the ductility ratio θ_{max}/θ_p under negative bending is 4.06 (150th reading).

4.4.3 LOAD-DEFLECTION RELATIONSHIPS

Load-deflection hysteresis loops of the specimen are

shown in Figures 3.9a and 3.9b. Maximum upward load attained in the test was 132.5 kips at the 150th reading.

This load is higher than the theoretical upward plastic load of 117.89 kips by 12%. The corresponding deflection at the 150th reading is not available because the dial gage malfunctioned. However, it is possible to derive approximate value of deflection from the reading of MTS display panel. By doing so, the deflection at the 150th reading is found to be 1714×10^{-3} inch. From equation 4.6, the theoretical upward yield deflection is found to be 326.5×10^{-3} inch. The deflection ductility therefore can be expressed as:

$$\frac{\Delta_{\max}}{\Delta_y} = \frac{1714}{326.5} = 5.25$$

The specimen showed good energy dissipation under both positive and negative moments. The specimen failed under an ultimate negative moment of 303.35 kips-ft at the face of the column stub. The true plastic moment capacity for this specimen was calculated as 270.78 kips-ft. as shown in Table 3.1. The ultimate moment was thus 12% higher than the true plastic moment. This improved performance as compared to Specimen SB2 was achieved because of the prevention of premature buckling of beam flanges and web due to the presence of longitudinal web stiffeners.

4.5 GENERAL BEHAVIOR

Hysteresis loops obtained from all the three specimens are stable, which show the high load reproduceability of the specimens. The results of the test on Specimen SB1 indicate that the steel specimen was able to sustain an ultimate load of at least 13% higher than its theoretical plastic load. The data obtained from the test of Specimen SB2 show that the specimen was barely able to reach its theoretical plastic moment. It could not take any further load due to buckling at the web of the beam. The beam section used for the specimen satisfied the width-to-thickness ratio prescribed in the current Canadian Code for Steel Structures CSA S16 for the flanges of plastic design sections, but failed to meet the corresponding criterion for the web. It appears that a less slender web would have made it possible to realize and exceed the negative plastic moment capacity of the composite section. A similar improvement in the capacity could be expected by the welding of stiffeners on the web. This was confirmed by the results of the test on Specimen SB3 which was able to withstand a load 12% higher than the theoretical plastic load.

The reloading paths of all the hysteresis loops of Specimens SB2 and SB3 show a pinching effect. This effect is caused by the closing and opening of cracks in the concrete slab. When the composite section is subjected to a negative moment, cracks are formed in the concrete slab. These cracks

open further as the longitudinal steel reinforcement yields. Yielding causes a permanent set in the steel. The cracks in the concrete do not therefore close up immediately upon the reversal of load, the stiffness of the section is thus lower. As the load is further increased, the cracks start to close and the stiffness of the section shows a sudden increase. This change in stiffness causes a change in the slope of the reloading paths on the hysteresis loops. Figure 4.3 shows the pinching effect on an idealized curve.

A summary of ductility ratios achieved during the tests is presented in Table 4.1. From the table, it can be deduced that Specimen SB1 had a slightly higher ductility ratio than that of Specimen SB2 in terms of curvature, rotation and deflection. On the other hand when compared with Specimen SB2 Specimen SB3 demonstrated a drop in curvature ductility but an increase in both rotation and deflection ductility. This is due to a higher concentration of curvature at the joint of Specimen SB2 as compared to that of Specimen SB3. Figure 4.4 shows the different curvature distribution patterns for Specimens SB2 and SB3. Due to the interaction between the horizontal web stiffeners and the web, Specimen SB3 has a wider yield area and as a result a lower localised increase in curvature.

Rotation and deflection can be expressed mathematically as:

$$\text{Rotation} = \theta = \int (M/EI) dx$$

$$\text{Deflection} = \Delta = \iint (M/EI) dx dx$$

where M/EI is the curvature distribution.

From the above expressions, it can be seen that rotation is proportional to the area under the M/EI curve while the deflection is proportional to the first moment of the area. Figure 4.4 shows the distribution of maximum curvature along the half length of Specimen SB2 and SB3. Value of the curvature obtained from test data are plotted at 3" and 9" from the face of the column. The curvature at elastic limit ϕ_y is obtained by the expression:

$$\phi_y = \frac{C_y}{y_c} \dots\dots\dots(4.7)$$

where C_y = yield strain of steel,

y_c = position of neutral axis.

The position of ϕ_y is obtained by first evaluating M_y by the expression:

$$M_y = \frac{F_y I'}{y_c} \dots\dots\dots(4.8)$$

where F_y = yield stress of steel,

I' = moment of inertia of composite section,

and then by linear proportion with the value of M_{ult} obtained from the test. It can be observed from Figure 4.4 that area under the M/EI curve for Specimen SB3 is greater than that for Specimen SB2, as a result the rotation and deflection ductilities of Specimen SB# are greater.

4.6 STRAIN DISTRIBUTION IN LONGITUDINAL REINFORCEMENT

Strain distribution in the longitudinal reinforcement across the width of the slab at 8" from the midspan are presented in Figures 3.10a and 3.10b for Specimen SB2 and in Figure 3.11a and 3.11b for Specimen SB3. The strains in the bars near the longitudinal centre lines of the beams are higher than those in the bars near the edge showing the shear lag phenomenon. All longitudinal reinforcing bars except those located near the edges of the specimens were yielding at the time the ultimate load was reached. This coupled with the fact that the specimens reached the plastic moment capacity evaluated on the assumption that all bars within the code prescribed effective width of slab were yielding shows that code provisions for effective width can be used in calculating the negative moment capacity of composite sections.

SPECIMEN	CURVATURE DUCTILITY	ROTATION DUCTILITY	DEFLECTION DUCTILITY
SB1	8.14	3.27	4.74
SB2	8.03	3.00	4.08
SB3	5.71	4.06	5.25*

*Approximation

TABLE 4.1

FIGURE 4.1 - IDEALISED MOMENT-ROTATION & MOMENT-CURVATURE RESPONSE.

B.M. Diagram

FIGURE 4.2

FIGURE 4.3

CHAPTER V

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

5.1 SUMMARY

The objective of this thesis was to investigate the load-deformation characteristic and ductility capacity of steel-concrete composite beam-to-column joint subject to cyclic-reversed loading. Three specimens of which one was base steel and the other two of composite sections were tested under different loading patterns. The composite sections were designed for two different negative moment capacities by varying the longitudinal reinforcement. In the specimen with a higher design negative moment, the web of the beam was strengthened by welding plate stiffeners.

Premature failure due to local buckling did not occur in any of the specimens. All specimens failed at a load higher than or equal to the plastic load. Decline of load carrying capacity was gradual after the development of local buckling in compression flange and web of the steel beam.

5.2 CONCLUSIONS

Unless mentioned otherwise, the following conclusions can only be applied directly to composite beams with dimension and material similar to the two composite specimens tested and subjected to similar form of cyclic loading.

Based on the results of the present investigation, the following conclusions can be drawn:

(1) Steel or steel-concrete composite sections exhibit stable hysteresis loops when subjected to cyclic-reversed loading. In the case of composite sections pinching effect exists on the reloading paths of the hysteresis loops due to the opening and closing of cracks in concrete slab.

(2) Unless the steel section used has fairly stocky flanges and webs, full negative plastic moment may not be realised due to premature local buckling of web and flange of the steel beam.

(3) Premature buckling of web and flange of the steel beam in a composite section subjected to negative moment can be avoided by welding longitudinal stiffeners on the web in the zone of plastic hinge. Sections that have been stiffened as mentioned above can realise their full plastic moment capacity under negative bending. In fact the ultimate moment is likely to be higher than the theoretical plastic moment due to strain hardening of steel. Such sections also exhibit good ductility capacity under cyclic loading.

(4) Most of the longitudinal reinforcement placed within a width of slab equal to the effective width determined for a composite section according to CSA Standard S16, 1969, clause 21.3.1 is yielding when the composite section is subjected to its ultimate negative moment. The effective width

provisions in the code can therefore be used when calculating the theoretical negative plastic moment of a composite section.

5.3 RECOMMENDATIONS

On the basis of this investigation, it has been possible to identify certain areas in which there is scope for further study. The topics that have the potential for further study are described below.

5.3.1 LOADING

The form of cyclic loading used in the present study had produced moment of the same sign on both sides of the column stub as shown in Figure 5.1. However, the distribution of bending moment that will occur in a building frame when subjected to earthquake forces is shown in Figure 5.2. The moments on two sides of an interior column are of different signs. Also the slab at the exterior column is discontinuous. Studies need to be carried out on the cyclic load behavior of interior beam-to-column joints where the moments on the two sides of the joint are of different signs. The effect of composite reinforced concrete slab on the behavior of an exterior beam-to-column joint should also be studied.

5.3.2 SECTION PROPERTIES

Specimens tested in the present study had the same width.

thickness (b/t) ratio for flange and also the same depth-to-thickness (h/w) ratio for web. The effect of varying b/t and h/w ratios on the cyclic load behavior needs further investigation.

5.3.3 CONSTRUCTION

Further investigation may also be carried out on the cyclic load behavior of different form of composite floor construction, such as for example a composite steel-concrete construction with the slab supported on a metal deck.

FIGURE 5.1

COMBINED MOMENT DISTRIBUTION OF EARTHQUAKE FORCES
DEAD LOAD.

FIGURE 5.2

LIST OF REFERENCES

- [1] R.W. Clough and Joseph Penzien; "Dynamics of Structures" - McGraw-Hill Book Company. 1975.
- [2] A.F. Lukey and P.F. Adams; "Rotation Capacity of Beams under Moment Gradient" - Journal of the Structural Division, ASCE, Vol. 95, No. ST6, June 1969, pp. 1173-1188.
- [3] Vitelmo V. Bertero, Egor P. Popov and Helmut Krawinkler; "Beam-Column Subassemblages Under Repeated Loading" - Journal of the Structural Division, ASCE, Vol. 98, No. ST5, May 1972, pp. 1137-1159.
- [4] Egor P. Popov and Vitelmo V. Bertero; "Cyclic Loading of Steel Beams and Connections" - Journal of the Structural Division, ASCE, Vol. 99, No. ST6, June 1973, pp. 1189-1204.
- [5] G.L. Agrawal, Leonard G. Tulin and Kurt H. Gerstle; "Response of Doubly Reinforced Concrete Beams to Cyclic Loading" - Journal of the American Concrete Institute, Title No. 62-51, Proc. V.62, No. 7, July 1965, pp. 823-834.
- [6] Robert Park, Dudley Charles Kent and Richard A. Sampson; "Reinforced Concrete Member with Cyclic Loading" - Journal of the Structural Division, ASCE Vol. 98, No. ST7, July 1972, pp. 1341-1360.
- [7] Mehmet Celebi and Joseph Penzien; "Experimental Investigation into the Seismic Behavior of Critical Regions of Reinforced Concrete Components as influenced by Moment and Shear" - National Technical Information Service, U.S. Department of Commerce. January 1973.
- [8] Barnard, P.R. and Johnson R.P.; "Plastic Behavior of Continuous Composite Beams" - Proceedings, The Institution of Civil Engineers, Vol. 32, October 1965, pp. 180-197.

- [9] Johnson, R.P., Finlinson, J.C.H. and Heyman, J.; "A Plastic Composite Design" - Proceedings, The Institution of Civil Engineers, Vol. 32, Paper No. 6892, October 1965, pp. 197-210.
- [10] Piepgrass, E.B. "Behavior of Composite Beams in Negative Bending" - Masters Thesis, Dept. of Civil Engineering, University of Alberta, June 1968.
- [11] Davison, J.H. and Longworth, J.; "Composite Beams in Negative Bending" - Structural Engineering Report No. 20, Department of Civil Engineering, University of Alberta, May 1969.
- [12] J. Hartley Daniels, G.D. Kroll and J.W. Fisher; "Behavior of Composite-Beam to Column Joints" - Journal of the Structural Division, ASCE, Vol. 96, No. ST3, March 1970, pp. 671-685.
- [13] J. Jay Climenhaga and R. Paul Johnson; "Local Buckling in Continuous Composite Beams" - The Structural Engineer, Vol. 50, No. 9, September 1972.
- [14] J.W. Fisher, J.H. Daniels and R.G. Slutter; "Continuous Composite Beams for Bridges" - Preliminary Report, International Association for Bridge and Structural Engineering, Ninth Congress, Amsterdam, May 1972, pp. 113-123.
- [15] R.P. Johnson and M. Hope-Gill; "Semi-Rigid Joints in Composite Frames" - Preliminary Report, International Association for Bridge and Structural Engineering, Ninth Congress, Amsterdam, May 1972, pp. 133-144.
- [16] J. Solnes; "Engineering Seismology and Earthquake Engineering".

DESIGN CALCULATIONS

I. DESIGN CALCULATIONS FOR SPECIMEN SB2

(1) POSITIVE MOMENT - (Using W12x27)

Local buckling check:-

$$\text{Flange : } \frac{b}{t} = \frac{3.25}{0.4} = 8.125 > 8.132 (= 54/\sqrt{44})$$

$$\text{Web : } \frac{h}{w} = \frac{10.125}{0.237} = 42.72 > 38.438 (= 255/\sqrt{44}) \text{ acceptable}$$

$$0.85 \times 3000 \times 60 \times a = 7.95 \times 44 \times 10^3$$

$$a = \frac{7.95 \times 44 \times 10^3}{0.85 \times 3000 \times 60} = 2.286" < 4.0"$$

$$l_p = \frac{11.96}{2} + (4 - \frac{2.286}{2}) = 8.837"$$

$$M_p = F_y \times A_s \times l_p$$

$$= 44 \times 10^3 \times 7.95 \times 8.837 \times \frac{1}{12} \times \frac{1}{10^3} = 257.59 \text{ kips-ft.}$$

$$M_{ps} = 139 \text{ kips-ft.}$$

$$P = \frac{257.59 \times 4}{10} = 103 \text{ kips.}$$

$$E_s = 29000 \text{ ksi} \quad E_c = W^{1.5} \frac{33}{\sqrt{f_c'}} = 145^{1.5} \frac{33}{\sqrt{3000}} = 3166 \text{ ksi}$$

$$m = 29000/3166 = 9.16 \quad \text{say } 9.$$

$$\begin{aligned} m A_s (d - h_c) &= 9 \cdot 7.95 (9.98 - 4) = 427.87 \\ \frac{1}{2} b_1 h_c^2 &= \frac{1}{2} \cdot 60 \cdot 4^2 = 480 \end{aligned} \quad \left. \vphantom{\begin{aligned} m A_s (d - h_c) &= 9 \cdot 7.95 (9.98 - 4) = 427.87 \\ \frac{1}{2} b_1 h_c^2 &= \frac{1}{2} \cdot 60 \cdot 4^2 = 480 \end{aligned}} \right\} x_1 < h_c$$

$$\begin{aligned} m A_s (d - x_1) &= \frac{1}{2} b_1 x_1^2 \\ 9 \times 7.95 (9.98 - x_1) &= \frac{1}{2} \times 60 \times x_1^2 \\ x_1 &= 3.8275'' \end{aligned}$$

$$\begin{aligned} I &= \frac{bx^3}{3m} + I_s + A_s (d - x_1)^2 \\ &= \frac{60 \times 3.8275^3}{3 \times 9} + 204 + 7.95 (9.98 - 3.8275)^2 \\ &= 629.53 \text{ in.}^4 \end{aligned}$$

Deflection check:-

$$\begin{aligned} \Delta_{\text{elastic}} &= \frac{C'}{10^6} \cdot M L_1^2 \left(\frac{6200}{I} \right) \\ &= \frac{0.8}{10^6} (257.59) (10^2) \left(\frac{6200}{629.53} \right) = 0.203'' \end{aligned}$$

(2) NEGATIVE MOMENT - (Using W12x27)

Make $M_p' = 0.75 M_p = 0.75 \times 257.59 = 193.19 \text{ kips-ft.}$

$$A_r f_y' = 2 f_y w dw_c \dots\dots\dots (1)$$

$$M_p' = M_{ps} + A_r f_y' (d_{sc} - d_r) \dots\dots\dots (2)$$

$$d_{sc} = \left(d - \frac{dw_c}{2} \right) \quad d_r = 2.0''$$

From (2): $(193.1925 \times 12) = (139 \times 12) + 2 f_y w dw_c (d_{sc} - d_r)$
 $650.31 = 2 \times 44 \times 0.237 \times dw_c (9.98 - 2 - 0.5 dw_c)$
 $dw_c = 6.83''$

$$> \frac{11.96}{2} - 0.4 = 5.58''$$

That is: N.A. lies in flange, therefore use different formula.

STRESS DIAGRAM

$$A_r f_y' = 2F_y.W.(\frac{d'}{2} - t) + 2F_y.b.(t - x) \dots\dots\dots(1)$$

$$M_p' = M_{ps} + 2F_y.W(\frac{d'}{2} - t) (l_1) + 2F_y b(t - x) (l_2) \dots(2)$$

$$M_{ps} = 139 \text{ kips-ft.} \quad d' = 11.96$$

$$M_p' = 193.19 \text{ kips-ft.} \quad (\frac{d'}{2} - t) = 5.58''$$

$$l_1 = (\frac{d'}{2} - t) \frac{1}{2} + t + (hc - dr) = \frac{5.58}{2} + 0.4 + 2 = 5.19''$$

$$l_2 = \frac{(t - x)}{2} + x + (hc - dr) = \frac{0.4}{2} + 0.5x + 2 = (2.20 + 0.5x)''$$

From (2):-

$$(193.19 \times 12) = (139 \times 12) + (2 \times 44 \times 0.237 \times 5.58 \times 5.19) + \\ (2 \times 44 \times 6.5(t-x) (2.2 + 0.5x)) \\ x = 0.366''$$

From (1):-

$$A_r \times 60 = (2 \times 44 \times 0.237 \times 5.58) + (2 \times 44 \times 6.5 \times 0.034) \\ A_r = 2.263 \text{ in.}^2$$

$$\text{Distribution steel @ } 0.5\% = \frac{0.5}{100} \times 12 \times 4 = 0.24 \text{ in.}^2/\text{ft.}$$

Upward deflection check:-

$$A_r (x-2) = A_s (d-x)$$

$$A_r (\text{provided}) = 2.45 \text{ in}^2$$

$$2.45 (x-2) = 7.95 (9.98-x)$$

$$x = 8.10''$$

$$I' = I_s + A_s (d-x)^2 + A_r (x-d_r)^2$$

$$= 204 + 7.95 (9.98-8.10)^2 + 2.45 (8.10-2)^2$$

$$= 323.26 \text{ in}^4$$

$$\text{elastic} = \frac{C'}{10^6} \cdot M L_1^2 \left(\frac{6200}{I'} \right)$$

$$= \frac{0.8}{10^6} \times 193.125 \times 10^2 \left(\frac{6200}{323.26} \right)$$

$$= 0.296''$$

II. DESIGN CALCULATIONS FOR SPECIMEN SB3

(1) POSITIVE MOMENT - (Using W12x27)

Note: Calculations similar to Specimen SB2.

(2) NEGATIVE MOMENT - (Using W12x27)

With reference to stress diagrams presented previously for Specimen SB2:-

$$\text{Design } M_p' = 0.85 M_p = 0.85 \times 257.59 = 218.95 \text{ kips-ft.}$$

$$l_2 = 2.2 + 0.5x \qquad l_1 = 5.19''$$

From (2):- (For N.A. lies within flange.)

$$\begin{aligned} 218.95 \times 12 &= (139 \times 12) + (2 \times 44 \times 0.237 \times 5.58 \times 5.19) + \\ &\quad (2 \times 44 \times 6.5 (t - x) (2.2 + 0.5x)) \\ \therefore x &= 0.125'' \end{aligned}$$

From (1):-

$$\begin{aligned} A_r \times 60 &= (2 \times 44 \times 0.237 \times 5.58) + (2 \times 44 \times 6.5 \times 0.275) \\ A_r &= 4.56 \text{ in.}^2 \end{aligned}$$

Distribution steel: @ 0.5% of web area

$$= \frac{0.5}{100} \times 12 \times 4 = 0.24 \text{ in}^2/\text{ft.}$$

III. DESIGN CALCULATIONS FOR SHEAR CONNECTORS
(SB2 AND SB3 SIMILAR)

From Table 8, CISC, capacity of 3/4" welded stud in
3000 psi concrete = 23 kips.

$$V_n = A_s F_y = 7.95 \times 44 = 349.8 \text{ kips.}$$

Total number of shear connectors required

$$= \frac{349.8}{23} \times 2 = 30.4 \quad \text{say, 32 numbers, i.e. 16 on each side of the beam.}$$

APPENDIX B

PERFORMANCE DATA OF MTS STRUCTURAL TESTING SYSTEM

A. Load

- (1) Maximum Load (force):

185,000 pounds tension/compression.

- (2) Full Scale Load Ranges:

±200 Kip

±100 Kip

± 50 Kip

± 20 Kip

- (3) Load Readout Accuracy:

Calibrated to within
1% of reading or 0.1% of range, whichever is
greater, but not greater than 0.5% of range.

- (4) Load Control Accuracy:

Static: 1% of setting or 0.1% of range,
whichever is greater.

Dynamic: 1% of range (typical).

Load Hold: 0.05% of range.

B. Stroke

- (1) Maximum Stroke:

6 inches

- (2) Full Scale Stroke Ranges:

±5.0 inches

±2.5 inches

±1.0 inches

±0.5 inches

(3) **Stroke Readout Accuracy:**

1% of range.

(4) **Stroke Control Accuracy:**

Static : 1% of range.

Dynamic : 1% of range (typical).

C. **Actuator Performance**

Maximum Ram Velocity:

115 inches/minute.

APPENDIX C

EVALUATION OF ACTUAL PLASTIC MOMENT AND ROTATION CAPACITY

I. CALCULATION OF TRUE M_p' FOR SPECIMEN SB2

$$A_r \text{ (provided)} = 2.45 \text{ in.}^2 \text{ (8 \#5 bars)}$$

$$\begin{aligned} f_y' &= \text{Actual yield stress of reinforcement} \\ &= \underline{52.168} \text{ ksi} \end{aligned}$$

$$\begin{aligned} F_y &= \text{Actual yield stress of structural steel} \\ &= \underline{48.22} \text{ ksi} \end{aligned}$$

$$A_r f_y' = 2F_y W\left(\frac{d'}{2} - t\right) + 2F_y b(t - x)$$

$$\begin{aligned} (2.45 \times 52.168) &= (2 \times 48.22 \times 0.237 \times 5.58) + \\ &\quad (2 \times 48.22 \times 6.50 \times (t - x)) \end{aligned}$$

$$127.81 = 127.53 + 626.86 (0.4 - x)$$

$$656.86(0.4 - x) = 0.28$$

$$4 - x = 0.0004466 \quad x = 0.399" \text{ within flange.}$$

$$1_1 = 5.19", \quad 1_2 = \left(2.2 + \frac{x}{2}\right) = 2.399" \quad (\text{see page A4})$$

$$(t - x) = 0.4 - 0.399 = 0.0004"$$

$$M_{ps} = F_y Z_p = 48.22 \times 37.90/12 = 152.29 \text{ kips-ft.}$$

$$\begin{aligned} M_p' &= M_{ps} + 2F_y W\left(\frac{d'}{2} - t\right)1_1 + 2F_y b(t - x)1_2 \\ &= (152.29 \times 12) + (2 \times 48.22 \times 0.237 \times 5.58 \times 5.19) + \\ &\quad (2 \times 48.22 \times 6.50 \times 0.0004 \times 2.399) \\ &= 1827.48 + 661.92 + 0.601 \\ &= 2490 \text{ kips-in.} \\ &= 207.50 \text{ kips-ft.} \end{aligned}$$

II.

SPECIMEN SB3CALCULATION OF TRUE M_p'

$$A_r \text{ (provide)} = 4.81 \text{ in.}^2 \text{ (8 \# 7 bars)}$$

$$\begin{aligned} f_y' &= \text{Tensile yield stress of reinforcement (actual)} \\ &= \underline{62.23 \text{ ksi}} \end{aligned}$$

$$\begin{aligned} F_y &= \text{Tensile yield stress of structural steel (actual)} \\ &= \underline{56.24 \text{ ksi}} \end{aligned}$$

$$\left(\frac{d'}{2} - t\right) = 5.58", \quad b = 6.50", \quad t = 0.4", \quad l_1 = 5.19"$$

$$l_2 = (2.2 + 0.5x) = 2.2 + 0.097 = 2.297"$$

$$(t - x) = 0.4 - 0.194 = 0.206"$$

$$A_r f_y' = 2F_y \cdot W \left(\frac{d'}{2} - t\right) + 2F_y \cdot b(t - x)$$

$$\begin{aligned} 4.81 \times 62.23 &= (2 \times 56.24 \times 0.237 \times 5.58) + \\ &\quad (2 \times 56.24 \times 6.50 \times (0.4 - x)) \end{aligned}$$

$$299.3263 = 148.75 + 731.12(0.4 - x)$$

$$731.12(0.4 - x) = 150.576$$

$$0.4 - x = 0.206$$

$$x = 0.194" \text{ within flange.}$$

$$M_p' = M_{ps} + 2F_y W\left(\frac{d'}{2} - t\right)l_1 + 2F_y b(t - x)l_2$$

$M_{ps} = F_y Z_p$ where Z_p = Plastic modulus of the section.

from given M_{ps} with $F_y = 44$ ksi

$$\text{We have: } 12 \times 139 = 44 Z_p$$

$$\text{i.e. } Z_p = \frac{12 \times 139}{44} = 37.90 \text{ in.}^3$$

For $F_y = 56.24$ ksi

$$\text{Actual } M_{ps} = \frac{56.24 \times 37.90}{12} = 177.62 \text{ kips-ft.}$$

Equation (2):

$$M_p' = (177.62 \times 12) + (2 \times 56.24 \times 0.237 \times 5.58 \times 5.19) + (2 \times 56.24 \times 6.50 \times 0.206 \times 2.297)$$

$$= 2131.44 + 772.01 + 345.95$$

$$= 3249.4 \text{ kips-in}$$

$$= 270.78 \text{ kips-ft.}$$

III.

SPECIMEN SB1CALCULATION FOR θ_p

$$E_s = 31,520 \text{ ksi.}$$

$$M_p' = M_{ps} = 152.29 \text{ Kips-ft.}$$

$$I' = I_s = 204 \text{ in.}^4$$

$$\begin{aligned}\theta_p &= \frac{M_p' l}{EI'} = \frac{152.29 \times 12 \times (60 - \frac{9.75}{2})}{31,520 \times 204} \\ &= \underline{0.0156} \text{ radian}\end{aligned}$$

IV.

SPECIMEN SB2CALCULATION FOR θ_p

$$A_s = 7.95 \text{ in.}^2$$

$$A_r = 2.45 \text{ in.}^2$$

$$A_r(x - 2) = A_s(d - x)$$

$$2.45(x - 2) = 7.95(9.98 - x)$$

$$2.45x - 4.9 = 79.34 - 7.95x$$

$$10.4x = 84.24$$

$$x = \frac{84.24}{10.4} = 8.10"$$

$$(d - x) = 1.88", \quad (x - 2) = 6.10"$$

$$\begin{aligned} I' &= I_s + A_s(d - x)^2 + A_r(x - 2)^2 \\ &= 204 + 7.95(1.88)^2 + 2.45(6.10)^2 \\ &= 323.26 \text{ in.}^4 \end{aligned}$$

$$E_s = 31,520 \text{ ksi.}$$

$$\theta_p = \frac{M_p l}{EI'} = \frac{207.50 \times 12 \times (60 - \frac{9.75}{2})}{31,520 \times 323.26} = 0.0134 \text{ radian.}$$

V.

SPECIMEN SB3VALUES OF Θ_p

$$A_s = 7.95 \text{ in.}^2$$

$$A_r = 4.81 \text{ in.}^2$$

$$A_r(x - 2) = A_s(d - x)$$

$$4.81(x - 2) = 7.95(9.98 - x)$$

$$4.81x - 9.62 = 79.34 - 7.95x$$

$$12.76x = 88.96 \quad x = \frac{88.96}{12.76} = 6.97''$$

$$\therefore (d - x) = 3.01'', \quad (x - 2) = 4.97''$$

$$\begin{aligned} I' &= I_s + A_s(d - x)^2 + A_r(x - 2)^2 \\ &= 204 + 7.95(3.01)^2 + 4.81(4.97)^2 = 394.84 \text{ in.}^4 \end{aligned}$$

$$E_s = 25530 \text{ ksi (tested value)} \quad \text{Actual } M_p' = 270.78 \text{ kips-ft.}$$

$$\therefore \Theta_p = \frac{M_p' l}{EI'} = \frac{270.78 \times 12 \times (60 - \frac{9.75}{2})}{25530 \times 394.84}$$

$$= 0.0177 \text{ radian}$$

END

[REDACTED]

END