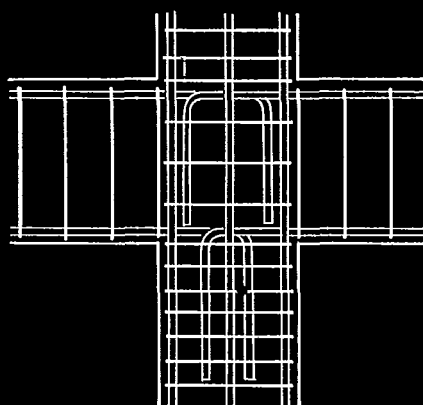
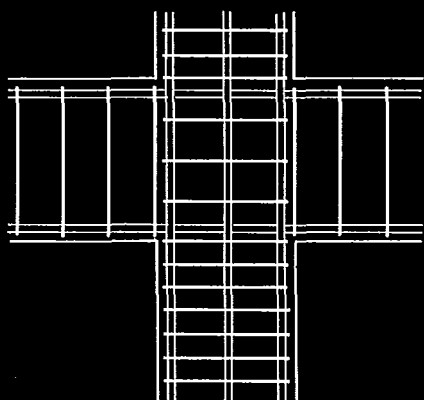
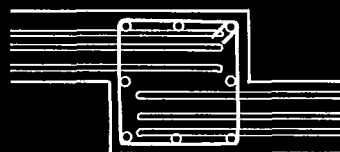
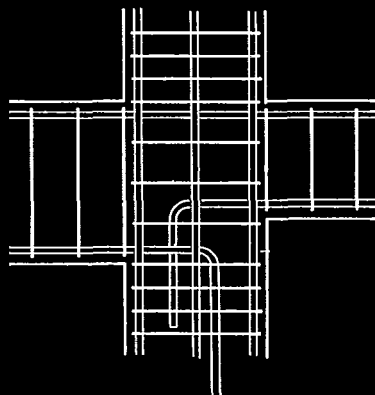
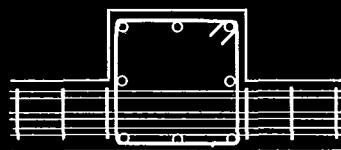
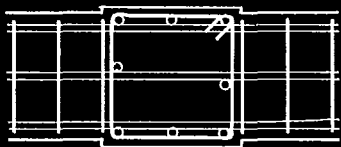
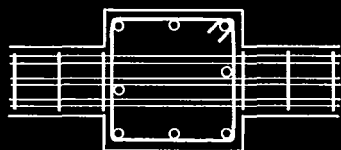


Design of Beam-Column Joints for Seismic Resistance

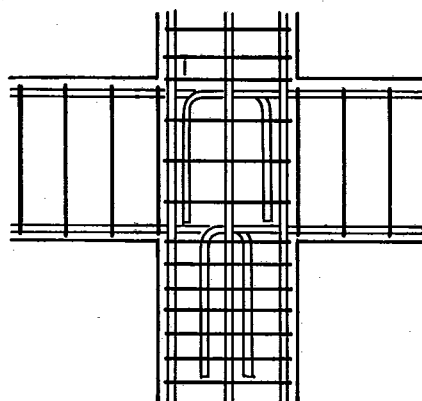
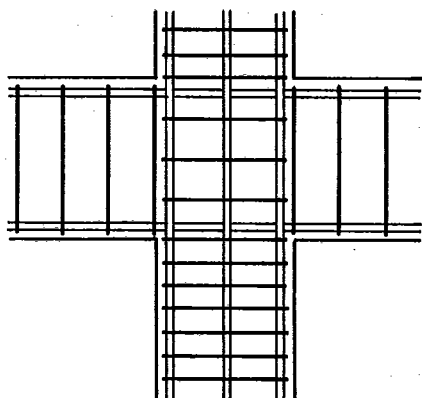
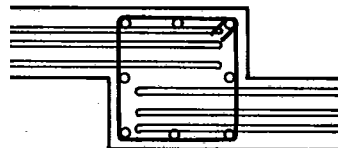
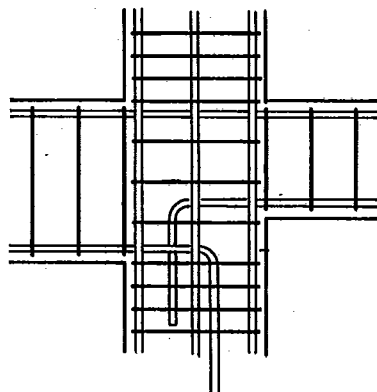
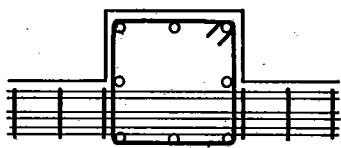
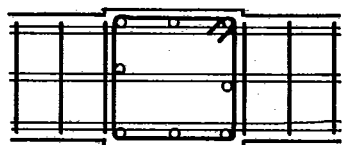
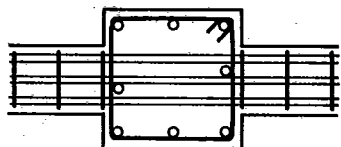


James O. Jirsa
Editor



AA Documents
SP-123
Document Sharing Hub

Design of Beam-Column Joints for Seismic Resistance



James O. Jirsa
Editor



AVA Documents
Document Sharing Hub
SP-123

DISCUSSION of individual papers in this symposium may be submitted in accordance with general requirements of the ACI Publication Policy to ACI headquarters at the address given below. Closing date for submission of discussion is April 1, 1992. All discussion approved by the Technical Activities Committee along with closing remarks by the authors will be published in the November/December 1992 issue of either ACI Structural Journal or ACI Materials Journal depending on the subject emphasis of the individual paper.

The Institute is not responsible for the statements or opinions expressed in its publications. Institute publications are not able to, nor intended to, supplant individual training, responsibility, or judgment of the user, or the supplier, of the information presented.

The papers in this volume have been reviewed under Institute publication procedures by individuals expert in the subject areas of the papers.

Copyright © 1991
AMERICAN CONCRETE INSTITUTE
P.O. Box 19150, Redford Station
Detroit, Michigan 48219

All rights reserved including rights of reproduction and use in any form or by any means, including the making of copies by any photo process, or by any electronic or mechanical device, printed or written or oral, or recording for sound or visual reproduction or for use in any knowledge or retrieval system or device, unless permission in writing is obtained from the copyright proprietors.

Printed in the United States of America

Editorial production Victoria Lunick

Library of Congress catalog card number 91-71710

PREFACE

During the past thirty years a great deal of research on beam-column joints and connections in reinforced concrete moment-resisting frames subjected to seismic loads has been conducted in structural testing laboratories all over the world. Based on these tests, design recommendations have been developed and incorporated into some codes.

In the United States the Committee on Joints and Connections in Monolithic Concrete Structures, sponsored jointly by the American Concrete Institute and the American Society of Civil Engineers (ACI/ASCE 352) first proposed design recommendations for reinforced concrete beam-column joints in a report published in 1976. The 1976 report was revised in 1985. The *"Building Code Requirements for Reinforced Concrete (ACI 318-83) - Appendix A: Special Provisions for Seismic Design,"* contained provisions for joint design which were based on work done by ACI/ASCE 352. The ACI 318 provisions formed the basis for beam-column joint design requirements in the Uniform Building Code and in the National Earthquake Hazards Reduction Program Recommended Provisions for Development of Seismic Regulations for New Buildings.

In New Zealand a *"Standard Code of Practice for the Design of Concrete Structures"* was published in 1982 by the Standards Association of New Zealand. This code contained a significant number of new design procedures, many of which were originated in New Zealand and were the result of New Zealand studies of the performance of reinforced concrete structures under earthquake conditions.

In Japan, the Architectural Institute of Japan has recently (1989) produced a document entitled, *"Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Ultimate Strength Concepts"* (In Japanese). Design provisions for beam-column joints for ductile reinforced concrete frames are provided.

A major concern over the years is that the recommended approaches for the design of beam-column joints in different countries may vary significantly. United States and New Zealand design procedures often lead to different amounts and arrangements of transverse reinforcement in beam-column joints. The differences are of major interest to designers, particularly since problems are often experienced during construction in physically placing the amount of reinforcement required by the codes in joint regions where congestion of reinforcement is likely.

After ACI/ASCE 352 published the 1976 report, designers became aware that, in spite of the rather large body of experimental data available, design recommendations were not consistent. Many of the differences could be traced to (1) interpretation of data obtained under different loading histories, (2) performance demanded of reinforced concrete beam-column joints in a moment resisting frame, (3) level of sophistication of design rules required for engineering practice, and (4) complexity and cost of construction resulting from proposed design procedures. The differences produced confusion in the application of design recommendations because the background was not explained adequately or explanations were not always available to the user.

An attempt to interpret the available test data in light of the differences in the current design rules was made by the Applied Technology Council. Project ATC-11 consisted of a study of design rules by a group of practicing engineers. The 1983 report, entitled *"Seismic Resistance of Reinforced Concrete Shear Walls and Frame Joints: Implications of Recent Research for Design Engineers"*, stated that

"A consensus needs to be reached on the most appropriate mechanism to represent the behavior of reinforced concrete frame joints under anticipated deformation reversals."

The ATC-11 report also included topics which required further research:

- Gross shear capacity of joints (including effects of column axial load)
- Joint confinement
- Orthogonal loading to produce multi-directional loading sequences
- Cyclic loading with high gravity loads
- Effect of eccentric beam-column connections.

In an effort to resolve some of the differences between codes and test interpretations, a group of researchers and designers met in Monterrey, California, following the 8th World Conference on Earthquake Engineering held in San Francisco, July 1984. Seventeen participants representing the U.S., Japan and New Zealand discussed the design approach used in each country and agreed to initiate a collaborative research effort to address the issues outlined in the ATC-11 report.

Following the Monterrey meeting, three universities - The University of Canterbury, University of Tokyo, and University of Texas at Austin - obtained funding to conduct a series of coordinated tests in which each group conducted tests of beam-column joint designs emanating from code provisions in their countries. The specimens were subjected to a common bidirectional load history. In 1985, researchers from Tongji University and The China Academy of Building Research initiated a test program which followed the general objectives of the coordinated project. A grant was made to the University of Texas by the National Science Foundation for U.S. participation in the cooperative project (Grant ECE-8320398).

To review the test program and to establish the common load history, a meeting was held in Tokyo in May 1985. The bi-directional load history selected is described in Papers 1-3. A special feature of that meeting was a session organized by Dr. Hiroyuki Aoyama at the Building Center of Japan in which a series of presentations were made by designers from construction firms on the general topic of *"Reinforced Concrete High-Rise Buildings in Japan with Special Emphasis on Aseismic Design of Beam-Column Joints."* Based on that seminar, a special report from the University of Texas was edited by Y. Kurose and J.O. Jirsa with funding from the National Science Foundation¹.

To review progress on the experimental studies, a meeting was held in Christchurch, New Zealand, in August 1987. At that meeting the participants agreed that the information being developed would be of considerable value to designers and code writers and it was decided that the information should be assembled and permanently recorded. The discussion led to the concept of an ACI Special Publication. A summary of recent Japanese research not readily available outside of Japan was edited by Y. Kurose and issued as a University of Texas report² following the meeting. Plans were also made to develop a final meeting which would involve both designers and researchers from the participating countries.

In 1989 a seminar was organized through the auspices of the National Science Foundation (Grant INT-8815956) and the Japan Society for the Promotion of Science. The *"Seminar on Earthquake Resistant Design of Reinforced Concrete Beam Column Joints"* was held May 24-26, 1989 at the East-West Center in Honolulu. Many of the papers presented at the

¹ "Proceedings of the Seminar on Reinforced Concrete High-Rise Buildings in Japan with Special Emphasis on Aseismic Design of Beam-Column Joints", Y. Kurose and J.O. Jirsa. PMFSEL Report No. 87-5, November 1987, University of Texas at Austin.

² Recent Studies on Reinforced Concrete Beam Column Joints in Japan," Y. Kurose. PMFSEL Report No. 87-8, December 1987, University of Texas at Austin.

Seminar are included in this volume. To further strengthen the transfer of information from research to practice, two technical sessions were sponsored by ACI 352 at the Fall 1989 ACI Convention in San Diego.

The participants (See list at end of Preface) and organizers (H. Aoyama, S. Otani, R. Park and J.O. Jirsa) of the various meetings and seminars associated with this project trust that the Special Publication will provide an insight into the cooperative research effort. The project demonstrated that researchers working cooperatively can accomplish a great deal through a coordinated, focussed effort. The results have already had an impact on Code development in some of the participating countries. Design recommendations are discussed in the papers included in the SP.

The papers are divided into the following groups.

- Papers 1-4 Tests conducted on specimens designed using current codes but with the same general geometry and a specified loading history
- Paper 5 Design recommendations - Japan
- Papers 6-13 Influence of joint geometry on strength and deformation characteristics
- Papers 14-17 Influence of bond on joint performance
- Papers 18, 19 Joints in precast systems and with high-strength materials.

The editor wishes to thank the contributors to this Special Publication for their efforts in preparing, reviewing and revising the manuscripts. The execution of a cooperative research effort can only succeed if some funding is available to permit communication, planning, and review. In that regard, the support of the U.S. National Science Foundation, the Japan Society for the Promotion of Science, the US/NZ Cooperative Science Program, and the Chinese National Science Foundation was essential to the project and is gratefully acknowledged. The participants in the project, the seminars, and the workshops have learned a great deal about behavior and design requirements for joints in moment-resisting frames. We sincerely hope that the readers will benefit from the accumulated knowledge represented by the papers in this volume.

January 1991

James O. Jirsa
Editor

PARTICIPANTS IN MEETINGS

1. Monterrey, CA: July 30 - August 1, 1984
 - New Zealand: *D.L. Hutchinson, L.M. Megget, A.J. O'Leary, R. Park, T. Paulay*
 - Japan: *H. Aoyama, O. Joh, S. Morita, H. Noguchi, S. Otani, M. Wakabayashi*
 - United States: *C.W. French, J.O. Jirsa, J.P. Moehle, J.L. Strata, J.K. Wight, L.A. Wyllie, Jr., J.B. Scalzi (NSF), A.J. Eggenberger (NSF)*

2. Tokyo, Japan: May 29-30, 1985
 - New Zealand: *R. Park, T. Paulay*
 - Japan: *H. Aoyama, K. Minami, S. Morita, K. Ogura, S. Otani, T. Shibata, M. Wakabayashi*
 - United States: *J.O. Jirsa*

3. Christchurch, New Zealand: August 10-12, 1987
 - New Zealand: *P. Cheung, R. Fenwick, G. McKay, L. Megget, A. O'Leary, R. Park, T. Paulay*
 - Japan: *H. Aoyama, O. Joh, K. Kitayama, S. Morita, H. Muguruma, S. Otani, H. Noguchi, S. Ogawa*
 - United States: *J.O. Jirsa*
 - China: *Chen, Y.C.*

4. East-West Center, Honolulu, Hawaii: May 24-26, 1989
 - New Zealand: *D.C. Hopkins, R. Park, T. Paulay*
 - Japan: *H. Aoyama, H. Hatamoto, T. Ichinose, O. Joh, T. Kaku, K. Kitayama, J. Kobayashi, Y. Kurose, S. Minami, S. Morita, T. Nagashima, M. Noguchi, S. Otani, K. Yoshioka*
 - United States: *A.J. Durrani, C.W. French, P. Gergely, J.O. Jirsa, C.S. Kopczynski, M.E. Kreger, R.J. Leon, J.P. Moehle, M.A. Sozen, D.R. Strand, J.K. Wight, L.A. Wyllie, Jr.*

COMMITTEE 352
Joints and Connections in Monolithic Concrete Structures
Joint ACI/ASCE

Michael E. Kreger, Chairman

Mohammad R. Ehsani, Secretary

Fariborz Barzegar
John F. Bonacci
James R. Cagley
Marvin E. Criswell
Ahmad J. Durrani
Catherine W. French
Luis E. Garcia
Cary S. Kopczynski
Donald F. Meinheit
Jack P. Moehle

Clarkson W. Pinkham
Mehdi Saiidi
Mustafa Seckin
Bahram M. Shahrooz
Gene R. Stevens
Donald R. Strand
Sudhakar P. Verma
John W. Wallace
James K. Wight
Loring A. Wyllie, Jr.

CONTENTS

PREFACE	iii
---------------	-----

TESTS TO VERIFY OR DEVELOP CODE PROVISIONS

NEW ZEALAND TESTS ON FULL-SCALE REINFORCED CONCRETE BEAM-COLUMN-SLAB SUBASSEMBLAGES DESIGNED FOR EARTHQUAKE RESISTANCE by P.C. Cheung, T. Paulay, and R. Park	1
EVALUATION OF SLAB-BEAM-COLUMN CONNECTIONS SUBJECTED TO BIDIRECTIONAL LOADING by Y. Kurose, G.N. Guimaraes, L. Zuhua, M.E. Kreger, and J.O. Jirsa	39
BEHAVIOR OF EXTERIOR REINFORCED CONCRETE BEAM-COLUMN JOINTS SUBJECTED TO BI-DIRECTIONAL CYCLIC LOADING by Z. Bolong and C. Yuzhou	69
DEVELOPMENT OF DESIGN CRITERIA FOR RC INTERIOR BEAM-COLUMN JOINTS by K. Kitayama, S. Otani, and H. Aoyama	97

CODE DEVELOPMENT

THE ARCHITECTURAL INSTITUTE OF JAPAN (AIJ) PROPOSAL OF ULTIMATE STRENGTH DESIGN REQUIREMENTS FOR RC BUILDINGS WITH EMPHASIS ON BEAM-COLUMN JOINTS by S. Otani	125
---	-----

INFLUENCE OF JOINT GEOMETRY ON STRENGTH AND DEFORMATION CHARACTERISTICS

COMPARISON BETWEEN INTERIOR AND EXTERIOR RC BEAM-COLUMN JOINT BEHAVIOR by S. Fujii and S. Morita	145
--	-----

DUCTILITY ESTIMATION OF EXTERIOR
BEAM-COLUMN SUBASSEMBLAGES IN
REINFORCED CONCRETE FRAMES

by T. Kaku and H. Asakusa 167

INFLUENCE OF TRANSVERSE JOINT AND
BEAM REINFORCEMENT AND RELOCATION OF
PLASTIC HINGE REGION ON BEAM-COLUMN
JOINT STIFFNESS DETERIORATION

by O. Joh, Y. Goto, and T. Shibata 187

EFFECT OF FLOOR SLAB ON BEHAVIOR OF
SLAB-BEAM-COLUMN CONNECTIONS

by C.W. French and J.P. Moehle 225

MECHANISMS OF SLAB CONTRIBUTIONS IN
BEAM-COLUMN SUBASSEMBLAGES

by P.C. Cheung, T. Paulay, and R. Park 259

REINFORCED CONCRETE WIDE-BEAM-TO-COLUMN
SUBASSEMBLAGES SUBJECTED TO LATERAL LOAD

by H. Hatamoto, S. Bessho, and Y. Matsuzaki 291

BEHAVIOR OF REINFORCED CONCRETE BEAM-COLUMN
JOINTS WITH ECCENTRICITY

by O. Joh, Y. Goto, and T. Shibata 317

BEHAVIOR OF BEAM-COLUMN JOINTS USING
HIGH-STRENGTH MATERIALS

by S. Sugano, T. Nagashima, H. Kimura, and A. Ichikawa 359

INFLUENCE OF BOND ON JOINT PERFORMANCE

INTERACTION BETWEEN BOND AT BEAM BARS AND
SHEAR REINFORCEMENT IN RC INTERIOR JOINTS

by T. Ichinose 379

BOND AND ANCHORAGE OF BARS IN REINFORCED
CONCRETE BEAM-COLUMN JOINTS

by T. Kaku and H. Asakusa 401

**TOWARDS NEW BOND AND ANCHORAGE
PROVISIONS FOR INTERIOR JOINTS**

by R.T. Leon 425

**ANALYSIS OF BOND DETERIORATION PROCESS IN
REINFORCED CONCRETE BEAM-COLUMN JOINTS
SUBJECTED TO SEISMIC LOADING**

by T. Tada and T. Takeda 443

JOINTS IN PRECAST SYSTEMS

**EXPERIMENTAL STUDY ON PREFABRICATED
BEAM-COLUMN SUBASSEMBLAGES**

by K. Yoshioka and M. Sekine 465

**BEAM-COLUMN JOINTS IN PRECAST CONCRETE
CONSTRUCTION IN JAPAN**

by Y. Kurose, K. Nagami, and Y. Saito 493

SI (Metric) TABLES 515

INDEX 517

SP 123-1

New Zealand Tests on Full-Scale Reinforced Concrete Beam-Column-Slab Subassemblages Designed for Earthquake Resistance

by P.C. Cheung, T. Paulay, and R. Park

Synopsis: As part of a United States/New Zealand/Japan/China collaborative research project, interior and exterior beam-column joint subassemblages with floor slabs of prototype two-way and one-way reinforced concrete building frames were designed for earthquake resistance using the current New Zealand concrete design code, NZS 3101:1982. Three full-scale subassemblages as designed were constructed and tested under quasi-static cyclic loading which simulated severe earthquake actions. The overall performance of each subassemblage during the tests was satisfactory in terms of strength and ductility. The joint core and column remained essentially undamaged while plastic hinges formed in the beams. The strong column-weak beam behaviour sought in the design, desirable in tall ductile frames designed for earthquake resistance, was therefore achieved. Although the joint cores of the subassemblages remained in the elastic range, joint core shear deformations contributed significantly to the interstorey drifts. Also, a significant proportion of the slab bars in tension contributed to the negative moment flexural strength of the beams. The performance of the one-way joint was superior to the performance of the two way joints.

Keywords: Beams (supports); capacity, columns (supports); cyclic loads; ductility; earthquake-resistant structures; frames; hinges (structural); joints (junctions); reinforced concrete; slabs; tests

2 Cheung, Paulay, and Park

P.C. Cheung is a PhD student in the Department of Civil Engineering of the University of Canterbury, New Zealand. He had several years of structural design experience with consulting engineers in Hong Kong prior to undertaking PhD studies.

T. Paulay is an Emeritus Professor of Civil Engineering of the University of Canterbury, New Zealand. His major interest has been the design of ductile reinforced concrete buildings for earthquake resistance. Apart from several papers he is the coauthor of two books on reinforced concrete. Amongst several awards he was recipient of the Raymond C. Reese Research Award of the ACI in 1982 and 1983. He is an Honorary Member of ACI.

R. Park, is Professor and Head of the Department of Civil Engineering of the University of Canterbury, New Zealand. He is the author or coauthor of several technical papers and two books on aspects of reinforced concrete, particularly seismic design. He has received several awards, including the 1984 and 1989 Raymond C. Reese Research Award of the ACI. He is a Fellow of ACI.

INTRODUCTION

Three full-scale reinforced concrete beam-column joint subassemblages with floor slabs were designed in compliance with the current New Zealand concrete design code for ductile moment resisting frames and tested under simulated earthquake loading. The structural dimensions and the quasi-static cyclic loading history followed the guidelines of the United States/New Zealand/Japan/China collaborative research project on the seismic design of reinforced concrete beam-column joints, as agreed at meetings of the principal investigators of the project held in the United States in 1984, Japan in 1985, New Zealand in 1987 and the United States in 1989.

In New Zealand a strong column-weak beam concept is used in the design of tall ductile moment resisting frames. The required flexural strength of the plastic hinge regions in the beams and column bases is first determined using the combinations of design seismic and gravity loadings. A capacity design procedure is then used to calculate the required shear strengths of beams, columns and joints, and the required flexural strengths of columns, with the aim of ensuring that inelastic deformations occur only at the chosen plastic hinge regions (1,2,3). The design forces for shear and column flexure are derived considering the possible flexural overstrengths of the plastic hinge regions of the beams, based on the as detailed reinforcing layout and the possible strengths of the longitudinal reinforcing steel taking into account yield strengths greater than specified

and strain hardening. In addition, the effects of higher modes of vibration and of seismic loading acting concurrently along the two axes of the building are taken into account when determining the column actions. In the three subassemblages tested, plastic hinges were designed to form in the beams at the column faces and the beams, columns and joints were reinforced accordingly.

In the joint core of each of the three subassemblages, reinforcement was placed to resist the total design horizontal shear forces acting across the joint core, as is required by the New Zealand concrete design code (1) when the axial load level on the columns is less than $0.1f_c' A_g$. Thus a considerable quantity of horizontal hoops was necessary in each joint core. A smaller amount of transverse reinforcement would be required in the joint core if the ACI building code (4) was followed. The New Zealand code (1) requirement is based on the observation that, when the axial load on the column is low, at large inelastic cyclic displacements the shear resistance provided by the diagonal concrete compression strut across the joint core diminishes, while that by a truss mechanism formed by the joint core reinforcement becomes dominant (2,3,5,6,7). When the axial column load is large this degradation of the diagonal compression strut mechanism is not so marked. To examine the worst case of joint core behaviour, no axial compression was applied to the upper columns of the subassemblages tested.

An important feature in the New Zealand code (1,2) is that the longitudinal slab bars within a prescribed width of slab on each side of the column can be counted on to act as part of the negative moment tension reinforcement of the beam. To study this slab effect, the layout of the slab reinforcement in each of the subassemblages followed closely that of prototype construction.

The aim of the tests was to examine the behaviour during major earthquake loading of subassemblages designed according to the New Zealand concrete design code (1), for comparison with the results obtained from all four countries of the collaborative research project. A particular aspect of interest was the effects of the presence of transverse beams and floor slabs on the behaviour of the subassemblages. Previous tests in New Zealand (5,6,7) have involved beam-column joint subassemblages without floor slabs and primarily of plane (one-way) frames.

TEST PROGRAMME

Description of Test Units

Each of the three subassemblages tested represented types of beam-column joints in three-dimensional moment resisting frames of buildings. The three subassemblages were as follows:

- Unit 1D-I : Interior beam-column-slab joint of a one-way frame.
- Unit 2D-I : Interior beam-column-slab joint of a two-way frame.
- Unit 2D-E : Exterior beam-column-slab joint of a two-way frame.

4 Cheung, Paulay, and Park

Details of the three Units, as designed, are shown in Figs. 1 to 3. The Units simulated full-scale subassemblages of frames with 3.5 m (11.5 ft) interstorey height. The beam shear span provided in the Units was only about two-thirds of the distance to the midspan of an assumed prototype which had a beam span of 6 m (19.7 ft). Also, the cantilever span of the one-way slab of Unit 1D-I, which modelled the topping slab of a ribbed floor system, was slightly less than one-third of an assumed prototype which had a 6 m (19.7 ft) span between two frames.

The reinforcing details are shown in Figs. 1, 2 and 3. The reinforcement ratios are listed in Table 1 and the measured properties of the reinforcing steel and concrete are shown in Table 2 and 3.

Design Features of Test Units

The main features of the design of the Units are summarized below.

In all Units the longitudinal reinforcement in the webs of all east-west beams (east beam only in Unit 2D-E) were kept identical. This enabled a direct comparison of the behaviour of exterior (Unit 2D-E) and interior (Units 1D-I and 2D-I) joints.

The ratios of the diameter of the longitudinal beam bars to the column depth was $d_b/h_c = 1/25$ for all Units. The ratio of the diameter of longitudinal column bars to the beam depth was $d_b/h_b = 1/22.9$, $1/19.6$ and $1/19.6$ for Units 1D-I, 2D-I and 2D-E, respectively. The New Zealand code (1) requirement for these ratios is $d_b/h_c \leq 1/25$ and $d_b/h_b \leq 1/20$.

The design shear forces for the beams and columns of the Units were calculated assuming that all the longitudinal beam and slab bars in tension were stressed to 1.25 times the specified yield strength of the steel. That is, to $1.25 \times 275 \text{ MPa}$ ($1.25 \times 40 \text{ ksi}$) in these Units. Generally, the quantity of transverse reinforcement in the beams and columns was governed by the limitations on the maximum spacing of transverse reinforcement for concrete confinement and for lateral restraint of compression reinforcement.

The design shear forces for the beam-column joint cores were also calculated assuming that the effective longitudinal reinforcement in the beams was stressed to 1.25 times the specified yield strength of the steel. The design horizontal and vertical shear forces so calculated are listed as V_{jh} and V_{jv} in Table 4 and are compared with the shear resistance provided by the horizontal and vertical joint core reinforcement, V_{sh} and V_{sv} . It should be noted that the intermediate longitudinal bars of the column were considered to be effective in resisting vertical shear in the joint cores. For horizontal shear, the joint hoops present in each Unit provided resistance almost identical to the design shear force.

To predict the theoretical nominal (ideal) strengths of the Units, the nominal (ideal) flexural strengths of the beam sections were calculated using the measured material properties of the concrete and steel. The New Zealand code (1) method was adopted which, like the ACI code (4), assumes an equivalent rectangular compressive stress block for concrete with a mean stress of $0.85f'_c$ and a maximum concrete compressive strain of 0.003. Since the floor slabs and beams were cast monolithically, they were expected to act integrally as T-beams. For negative bending moment producing tension in the top bars, the New Zealand code (1,2) considers that all longitudinal bars placed within a specified flange width act as tension reinforcement for the beam. The flange width is of varying magnitude depending on the structural configurations, and is illustrated in Fig. 4. These New Zealand code recommendations were followed in the theoretical calculations. In addition, for T-beams subjected to positive bending moment under seismic conditions, the effective width of the flange in compression was assumed in all cases to be twice the column width. Calculations showed that for positive bending moment, larger slab widths did not increase the moment capacity significantly. Flexural strength values estimated using other assumptions have been discussed in Refs. 8 and 9. The ratio of column flexural strengths to beam flexural strengths, obtained using the New Zealand code assumptions for effective widths of slabs, are listed in Table 5.

The nominal (ideal) strength of each Unit subjected to seismic loading is attained when a positive moment plastic hinge occurs in the beam at one face of the column and a negative moment plastic hinge occurs in the beam at the other face of the column, as in Fig. 5(b). In the figures showing the measured lateral load-lateral displacement hysteresis loops for the Units (Figs. 12, 13 and 14), this theoretical ideal strength is denoted as V_i when the New Zealand code recommendations are followed for the calculation of the width of flange in tension contributing to the negative moment strength of the beam. For comparison, those figures also show a theoretical ideal strength denoted as V_i^* which is calculated assuming that the longitudinal slab bars over the full width of the slab yield in tension and contribute to the negative moment flexural strength of the beam.

Loading Rig and Test Procedure

The Units were extensively instrumented in order to gather as much information as possible on the load-displacement responses, joint distortions, beam curvatures and strains in the reinforcing bars. Full details of the instrumentation can be seen in Refs. 8 and 9.

Figure 5(b) shows the manner of loading the Units by displacing the beam ends. Figure 6 shows a schematic configuration of the loading rig, which was designed so as to apply either unidirectional or bidirectional (i.e. orthogonal east-west and north-south) simulated seismic loading. Pins were

6 Cheung, Paulay, and Park

provided in the two directions to enable the top and bottom ends of the columns of the Units to rotate in the loading directions. The beam ends, where the double-acting jacks imposed vertical loads, were also free to rotate and move laterally. Figure 7 shows a Unit during testing.

By considering the geometrical relationships shown in Fig. 5(b), the equivalent interstorey displacement Δ_c and interstorey shear V_c can be derived from the applied beam vertical loads, P_1 and P_2 , and corresponding displacements, Δ_{B1} and Δ_{B2} , using the following expressions:

$$\Delta_c = \left(\frac{\Delta_{B1} + \Delta_{B2}}{l_1 + l_2} \right) l_c \quad (1)$$

$$V_c = \left(\frac{P_1 l_1 + P_2 l_2}{l_c} \right) \quad (2)$$

where the notation is as shown in Fig. 5(b).

During the tests, continuous corrections to the overall measured displacements were necessary to allow for the flexibility of the steel loading rig which caused some horizontal movement of the top of the Units.

Loading Sequence

The "first-yield displacement" $\Delta_{y, \text{test}}$ was determined by extrapolation from the measured displacement at 75% of the ideal strength V_1 of the Unit, as shown in Fig. 8.

The New Zealand loadings code (10) permits a maximum interstorey drift (Δ/l_c in Fig. 5(a)) of 0.32% computed for the frame assuming elastic behaviour under the code seismic loading. The experimentally measured interstorey drifts when the displacement ductility factor was $\mu = 1$ (found from Δ/l_c , where Δ is given by Eq. 1 with Δ_{B1} and Δ_{B2} given by the first yield displacement as defined in Fig. 8) was 0.45% for Unit 1D-I, 0.47% for Unit 2D-I and 0.35% for Unit 2D-E.

The quasi-static cyclic loading histories followed for the three Units are depicted in Figs. 9 to 11. In each test, the first two load cycles were to an imposed lateral load of about one-half of V_1 . The first yield displacement $\Delta_{y, \text{test}}$, and the measured stiffness K_{test} , were determined in the first half of the third load cycle. Subsequent cycles were displacement controlled with increasing imposed displacement ductility factors $\mu = \Delta/\Delta_{y, \text{test}}$ and enabled observation of the performance of the Units at low as well as high ductilities. According to the New Zealand loadings code (10), a ductile structure should be able to undergo four cycles of loading to a displacement ductility factor of four in each direction, implying a cumulative displacement

ductility factor demand of $\Sigma\mu = 32$. Furthermore, after those four cycles, the reduction in strength of each individual component should not exceed 30%, while that of the whole structure should not exceed 20%.

For Unit 2D-E (Fig. 11), loading was applied principally in the east-west direction (that is, perpendicular to the spandrel beam). The values for $\Delta_{y, \text{test}}$ and K_{test} were obtained for when the east beam was displaced downwards and were significantly different from those found for Unit 2D-I. In order to make a more meaningful comparison of the behaviour of Unit 2D-E with that of the other two Units, the subsequent imposed displacements for Unit 2D-E were to similar interstorey drift levels as for the other two Units.

TEST RESULTS

General Observations

The experimental (measured) hysteresis loops for column lateral load (storey shear) versus lateral displacement are shown in Figs. 12 to 14. For Units 2D-I and 2D-E both the north-south and the east-west load-displacement responses are given.

All three Units performed extremely well during the tests and easily satisfied the New Zealand loadings code (10) performance criterion for a ductile structure. Plastic hinges formed in the beams at the column faces when the Units were loaded into the inelastic range. Each column developed only fine cracks and remained in the elastic range. Figures 15 to 17 show overall views of the Units during testing when large ductilities had been imposed. Although in the joint core regions diagonal cracks and sometimes spalling of concrete surfaces were observed, the joint cores remained essentially intact.

Lateral Load-Displacement Response

For all Units, although there was a gradual degradation of stiffness during the testing, the shape of the hysteresis loops remained reasonably stable up to interstorey drifts of at least 3%. For Unit 2D-I the particularly flat and pinched curve for run 36 up to an interstorey drift of 4% (see Fig. 13(a)) was caused by some slippage of the east-west bottom beam bars. In other cases the pinching of loops which occurred was mainly associated with some buckling of compressed beam bars after cover concrete had spalled. Deterioration of the Units became significant only when the interstorey drift was well in excess of 3%.

The theoretical strengths V_t of the Units, which assumes an effective flange width in tension equal to that recommended in the New Zealand code (see Fig. 4), were consistently exceeded during the tests. The sources of strength enhancement were mainly from the strain hardening of the reinforcing bars in

8 Cheung, Paulay, and Park

tension and from contributions of slab bars over a greater width than assumed in the New Zealand code. The theoretical maximum strength V_x , which assumes an effective flange width in tension equal to the entire width of slab, is also plotted in the figures. Evidently, since strain hardening of all the longitudinal reinforcement in the beams was inevitable, not all slab bars over the entire width of slab were contributing to the strength enhancement.

A feature of the hysteretic responses of the two-way Units 2D-I and 2D-E (see Figs. 13 and 14) was the reduction in peak load capacity by as much as 20% while the predetermined displacement ductility was being applied in the perpendicular loading direction. It is believed that apart from the effect of creep in the Units, this strength reduction was primarily caused by the changes in the contributions of the reinforcement in the slab to the flexural strengths of the beams while the loading was being applied in the perpendicular direction (11).

Comparison of the measured hysteretic responses of the three Units (Figs. 12 to 14) shows that the performance of the one-way interior (Unit 1D-I) was superior to that of the two-way exterior joint (Unit 2D-E), which in turn was superior to that of the two-way interior joint (Unit 2D-I). This observation is of interest since the joint regions of the three Units were similarly designed (Units 1D-I and 2D-I had identical transverse reinforcement). The effects of unidirectional and bidirectional loading, and of the presence of transverse beams and slab on the joints, are discussed in more detail elsewhere (11).

The contributions to the displacement of the Units were analysed as the sum of the deformations from the beams, columns and joint cores. In the tests, rotations in each beam over a length of 1.5 times the depth of beam from the column face were measured. This measurement included the plastic hinge rotations and the effect of beam bar deformations in and slip through the joint core. The elastic deformations over the remaining length of the beam and of the column were considered less significant and were estimated by calculations based on traditional elastic theory methods (5). For joint core shear distortions, experimental measurements were made only for Units 1D-I and 2D-E (north-south). Figure 18 shows the percentage contributions of these various deformation components for Unit 1D-I. Results for the other Units were similar. As expected, the major source of interstorey drifts were from beam plastic hinge deformations. Nevertheless, an important observation from Fig. 18 is that joint core shear deformations were significant. Joint core shear deformations accounted for about 26% of total interstorey drift at a displacement ductility factor $\mu = 2$. This proportion decreased to less than 20% at higher ductilities and became reasonably constant. Hence, although adequate transverse reinforcement was provided for the joint cores to remain mainly in the elastic range, it is evident that joints were far from rigid.

Strains in Reinforcing Bars

Typical strain variations in the horizontal joint hoops are illustrated in Figs. 19 and 20. The imposed ductilities are shown circled and a * on the ductility indicates bidirectional loading. Figures 19 and 20 indicate a gradual and consistent increase in tensile strains as the level of imposed ductility increased. Hence the role of the hoops in resisting joint core shear forces through a truss mechanism, and in providing confinement of the joint core, became more significant as the ductility levels increased. In particular, type E legs (Fig. 19) and type B legs (Fig. 20) exhibited larger strains than the other types, evidently because they were subjected to greater bond forces from the beam bars. It is also noted that the hoop strains increased during imposed bidirectional loading (compare, for example, results for $\mu = \pm 3$ and $\pm 3^*$ in Fig. 20). This would have been because, for example, the loading in the north-south direction would have caused dilation in the joint core with the formation of diagonal cracks. The east-west hoop legs were then required to provide the necessary containment, thus resulting in additional tensile strains. Hence the transverse beams did not provide significant confinement to the joint core. It is of interest that the joint core hoops generally did not reach yield. Yield was only reached at high displacement ductility factors in some hoops.

Strains were measured on the longitudinal beam and column bars in the joint cores. The measured strains indicated that those bars were adequately anchored. For the column bars away from the column corners of Unit 2D-I during unidirectional loading, tensile strains less than the yield strain but of significant magnitudes were measured within the joint core. The participation of these bars in resisting vertical joint shear, analogous to the horizontal hoops, is therefore evident. Considerably higher strains generated in the column bars under bidirectional loading indicate that the column bars in the joint core were subjected to a more severe tensile stress state during bidirectional loading.

Strains measured on longitudinal beam bars and the slab bars during the tests were converted to stresses using a computer program (12) based on the Ramberg-Osgood stress-strain model. Figure 21 shows for example the stresses in some slab bars of Unit 2D-I. It is significant that a large proportion of those slab bars have tensile stresses, and the width of the slab with bars in tension exceeds that recommended by the New Zealand concrete design code (1,2), shown in Fig. 4. The mechanisms of slab contributions to the behaviour of beam-column assemblies is discussed in a separate paper (11).

CONCLUSIONS

1. The performance of the three beam-column-slab subassemblages, Units 1D-I, 2D-I and 2D-E, which were designed

10 Cheung, Paulay, and Park

according to the New Zealand concrete design code, provisions for ductile moment resisting frames, was very satisfactory during the cycles of quasi-static cyclic loading which simulated severe seismic loading. The performance of the one-way interior joint (Unit 1D-I) was superior to that of the two-way exterior joint (Unit 2D-E), which in turn was superior to that of the two-way interior joint (Unit 2D-I).

2. The very good performance of the beam-column-slab subassemblages was considered to have resulted from the relatively large quantity of joint core shear reinforcement provided, and the use of sufficiently small diameter longitudinal beam bars to avoid excessive slippage through the joint cores when plastic hinges developed in the beams at the column faces.

3. There was no evidence during the tests to indicate that the presence of floor slabs or beams in two directions provided significant confinement to the joint cores during bidirectional seismic loading.

4. Although the beam-column joint cores remained essentially in the elastic range during the tests, the contributions of joint core shear deformations to the interstorey drift of each Unit was significant and was as high as 26% of the total interstorey drift, reducing to about 20% of the total at large ductilities.

5. A significant proportion of slab bars in tension contributed to the negative moment flexural strength observed in the beams and hence to the enhancement of the flexural strength of each Unit. The lateral load strength of the Units was up to 39% higher than that calculated using the effective flange widths in tension assumed in the New Zealand concrete design code. In this calculation the measured material strengths and a strength reduction factor ϕ of unity were used. Some of this strength enhancement was due to strain hardening of the steel.

6. The strengths and stiffnesses of the Units reduced when bidirectional loading was applied, due to changes in the contributions of slab reinforcement.

ACKNOWLEDGEMENTS

This research work was carried out in the Department of Civil Engineering of the University of Canterbury, New Zealand, as part of the United States/New Zealand/Japan/China collaborative research project on the seismic design of reinforced concrete beam-column joints. Financial support from the following organisations in New Zealand is gratefully acknowledged: the Building Research Association of New Zealand, the Ministry of Works and Development, the University Grants Committee, the University of Canterbury, and the US/NZ Cooperative Science Programme. Pacific Steel Ltd, Auckland, kindly provided the steel reinforcement.

REFERENCES

1. "Code of Practice for the Design of Concrete Structures, NZS 3101 Part 1:1982", Standards Association of New Zealand, Wellington, 1982, 127pp.
2. "Commentary on the Design of Concrete Structures, NZS 3101 Part 2:1982", Standards Association of New Zealand, Wellington, 1982, 156pp.
3. Park, R., "Ductile Design Approach for Reinforced Concrete Frames", Earthquake Spectra, Professional Journal of the Earthquake Engineering Research Institute, Vol. 2, No. 3, May 1986, pp.565-619.
4. ACI Committee 318, "Building Code Requirements for Reinforced Concrete (ACI 318-83)", American Concrete Institute, Detroit, 1983, 111pp.
5. Park, R. and Paulay, T., "Reinforced Concrete Structures", John Wiley & Sons, New York, 1975, 769pp.
6. Paulay, T., Park, R. and Priestley, M.J.N. "Reinforced Concrete Beam-Column Joints Under Seismic Actions", Journal of the American Concrete Institute, Proceedings, Vol. 75, No. 11, Nov. 1978, pp.585-593.
7. Park, R. and Milburn, J.R., "Comparison of Recent New Zealand and United States Seismic Design Provisions for Reinforced Concrete Beam-Column Joints and Test Results from Four Units Designed According to the New Zealand Code", Bulletin of New Zealand National Society for Earthquake Engineering, Vol. 16, No. 1, March 1983, pp.3-24.
8. Cheung, P.C., Paulay, T. and Park, R., "A Reinforced Concrete Beam-Column Joint of a Prototype One-Way Frame with Floor Slab Designed for Earthquake Resistance", Research Report 87-6, Department of Civil Engineering, University of Canterbury, Christchurch, July 1987, 52pp.
9. Cheung, P.C., Paulay, T. and Park, R., "Interior and Exterior Reinforced Concrete Beam-Column Joints of a Prototype Two-Way Frame with Floor Slab Designed for Earthquake Resistance", Research Report 89-2, Department of Civil Engineering, University of Canterbury, March 1989, 80pp.
10. "Code of Practice for General Structural Design and Design Loadings for Buildings, NZS 4203:1984", Standards Association of New Zealand, Wellington, 1984, 100pp.
11. Cheung, P.C., Paulay, T. and Park, R., "Mechanisms of Slab Contributions in Beam-Column Subassemblages", ACI Special Publication on Beam-Column Joints.

12 Cheung, Paulay, and Park

12. Carr, A.J., "Hysteresis", Computer Program Library, Department of Civil Engineering, University of Canterbury, Christchurch, 1985.

NOTATION

A_g	= gross area of column
A_{jh}	= total area of horizontal shear reinforcement crossing the joint core
A_{jv}	= total area of vertical shear reinforcement crossing the joint core
A_{st}	= total area of longitudinal reinforcement in column
b_c	= width of column
d_b	= diameter of reinforcing bar
f'_c	= compressive cylinder strength of concrete
f_{su}	= ultimate tensile strength of steel
f_y	= yield strength of steel
h_b	= beam depth
h_c	= column depth
h_s	= thickness of floor slab
K	= stiffness
l	= length of beam
l_c	= storey height, centre to centre between floors
P	= vertical load at beam end
V	= lateral load at column end
V_i	= theoretical shear force in column of Unit when plastic hinges develop in beams, using effective flange widths assumed in the New Zealand code
V_i^*	= theoretical shear force in column of Unit when plastic hinges develop in beams, assuming effective flange width in tension equal to the entire width of slab and effective flange width in compression as in the New Zealand code
V_{jh}	= horizontal shear force across joint core

- V_{jv} = vertical shear force across joint core
 V_{sh} = horizontal joint core shear strength provided by horizontal shear reinforcement
 V_{sv} = vertical joint core shear strength provided by vertical shear reinforcement
 Δ = horizontal displacement
 Δ^* = displacement observed when three-quarters of ideal strength is attained
 Δ_{B1}, Δ_{B2} = observed vertical displacement of beam end
 Δ_c = total horizontal displacement of test column
 Δ_e = computed interstorey drift for the structure acting elastically when subjected to design seismic loading
 $\Delta_{y, test}$ = first yield displacement
 δ = vertical displacement of beam end
 θ = angle of rotation
 μ = $\Delta / \Delta_{y, test}$ = displacement ductility factor
 ϕ = strength reduction factor

Table 1 - Summary of Ratios of Longitudinal Reinforcing Steel

Unit			1D-I	2D-I	2D-E
Slab	East/West	Top Steel	0.224%	0.337%	0.232%
		Bottom Steel	0.224%	0.252%	0.252%
	North/South	Top Steel	0.447%	0.377%	0.377%
		Bottom Steel	0.223%	0.252%	0.252%
Beam	East/West	Top Steel	1.34%	1.58%	1.34%
		Bottom Steel	0.77%	0.77%	0.77%
	North/South	Top Steel	-	1.57%	1.30%
		Bottom Steel	-	0.73%	1.03%
Column			1.48%	2.05%	2.69%

Notes: Reinforcement ratios of slab and column and expressed in terms of gross areas of slab and column. Reinforcement ratios of beams are expressed in terms of beam width and effective depth and top steel includes slab steel within New Zealand code effective width (see Fig. 4).

14 Cheung, Paulay, and Park

Table 2 - Measured Properties of Reinforcing Steel of Units

Grade of Steel	Longitudinal Reinforcement in Beams and All Transverse Reinforcement (Grade 275 Steel)							Longitudinal Reinforcement in Columns (Grade 380 Steel)			
	Bar Size	R10	R12	D10	R16	D16	D20	D24	HD20	HD24	HD28
Yield Strength f_y (MPa)		315	320	326	330	318	300	283	482	500	432
Ultimate Strength f_{su} (MPa)		432	466	441	503	482	459	437	650	669	602

Notes : 1 MPa = 145 psi

R10 = plain round bar of 10 mm diameter

D10 = deformed bar of 10 mm diameter

HD20 = deformed high strength bar of 20 mm diameter

Table 3 - Measured Properties of Concrete of the Units

Unit	1D-I		2D-I		2D-E	
	1	2	1	2	1	2
Pour						
Slump (mm)	75	150	50	100	75	150
f'_c at 28 Days (MPa)	43.5	25.3	35.5	25.1	43.5	25.3
Age at Test	80 days	71 days	130 days	126 days	80 days	71 days
f'_c at Test (MPa)	38.0	26.4	37.0	27.4	38.0	26.4

Notes : Pour 1 included lower column, beams, slab and joint; pour 2 was for upper column only.

1 mm = 0.0394 in

1 MPa = 145 psi

Table 4 - Comparison of Required Joint Shear Forces with Shear Resistance Provided by Reinforcement of Units According to the New Zealand Concrete Design Code

Unit	1D-I	2D-I	2D-E	
Horizontal shear force:		(EW & NS)	(EW)	(NS)
(a) Design V_{jh} (kN)	1212	1350	764	1041
(b) Provided V_{sh} (nominal) (kN)	1061	1061	678	874
(c) Provided V_{sh} (actual) (kN)	1273	1273	807	1046
Vertical shear force:				
(d) Design $0.4V_{jv}$ (kN)	444	518	479	
(e) Provided V_{sv} (nominal) (kN)	478	936	936	

Notes :

- (a) Design horizontal shear force is V_{jh} , where V_{jh} is given by the resultant horizontal force acting either above or below a horizontal plane at the mid-depth of the joint core. It is considered by the code that all of V_{jh} is to be resisted by the horizontal shear reinforcement. The nominal joint core shear stress $V_{jh}/b_c c$ was less than $1.5\sqrt{f'_c}$ MPa in all cases.
- (b) Provided (nominal) shear resistance of horizontal joint core reinforcement is $V_{sh} = A_{jh} f_y$, estimated assuming $f_y = 275$ MPa as specified.
- (c) Provided (actual) shear resistance of horizontal joint core reinforcement is $V_{sh} = A_{jh} f_y$, where f_y = measured yield strength of steel used in tests.
- (d) Design vertical shear force for vertical reinforcement is $0.4V_{jv}$, where V_{jv} is given by the resultant vertical force acting on one side of a vertical plane through the joint core. It is considered by the code that $0.4V_{jv}$ is to be resisted by the vertical shear reinforcement and $0.6V_{jv}$ is to be resisted by the concrete diagonal compression strut.
- (e) Provided (nominal) vertical shear resistance of intermediate column bars is $V_{sv} = A_{jv} f_y$, estimated assuming $f_y = 380$ MPa as specified.

1 MPa = 145 psi.

16 Cheung, Paulay, and Park

Table 5 - Comparison of Design and Provided Flexural Strengths of Columns on the Basis of Specified Material Properties

Unit	1D-I	2D-I		2D-E	
	EW	EW	NS	EW	NS
(1) M_{des} (kNm)	274.1	306.1	321.0	157.5	230.8
(2) M_{max} (kNm)	383.1	468.6	486.0	255.9	325.7
(3) M_{col} (kNm)	474.7	700.4	700.4	626.9	560.3
$\frac{M_{col}}{M_{des}}$	1.73	2.29	2.18	3.98	2.43
$\frac{M_{col}}{M_{max}}$	1.24	1.49	1.44	2.45	1.72

Notes : (1) M_{des} is the column moment when the positive and negative moment flexural plastic hinges form in the beams, with the effective tension flange width of the beam as assumed by the New Zealand code and tensile steel stress $f_y = 275$ MPa.

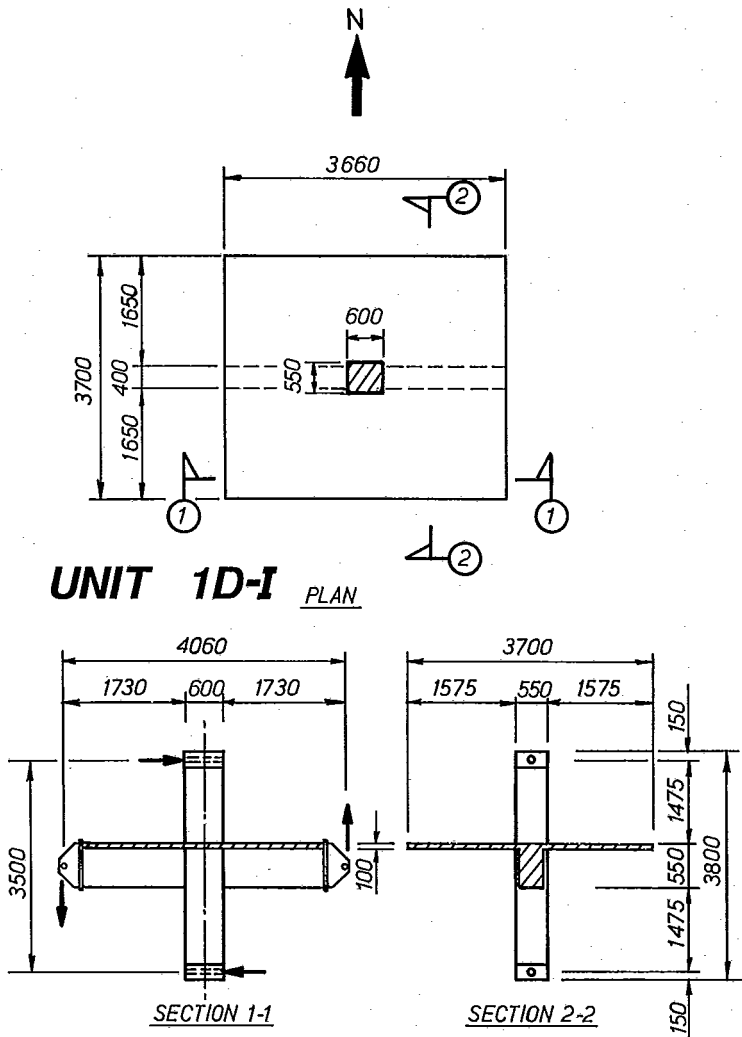
(2) M_{max} is the maximum column moment when all beam and slab bars in tension have reached strain hardening with steel stress as 1.25×275 MPa.

(3) M_{col} is the nominal (ideal) flexural of the column with steel stress as $f_y = 380$ MPa.

1 MPa = 145 psi

1 kNm = 0.738 kip-ft

$f'_c = 30$ MPa



NOTES:

1. Concrete cover to main reinforcement of beams and columns to be 40mm

2. Grades of reinforcing bars to be as follows:
R = Grade 275 round bars
D = Grade 275 deformed bars
HD = Grade 380 deformed bars

Fig. 1--Details of Unit ID-I

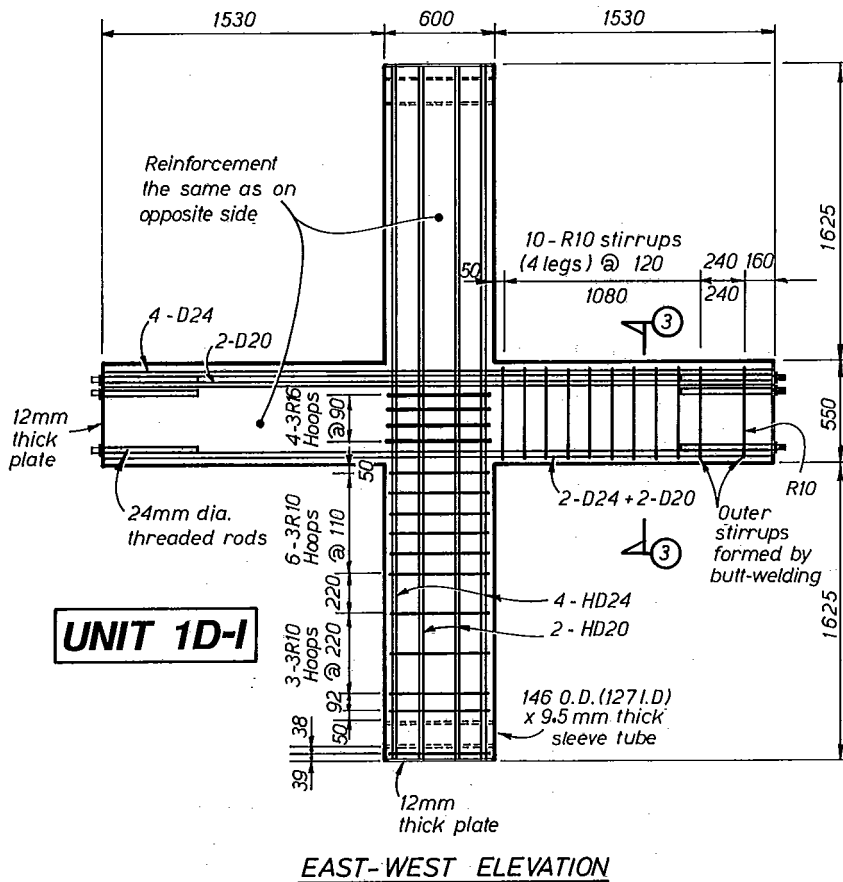


Fig. 1 cont.--Details of Unit ID-I

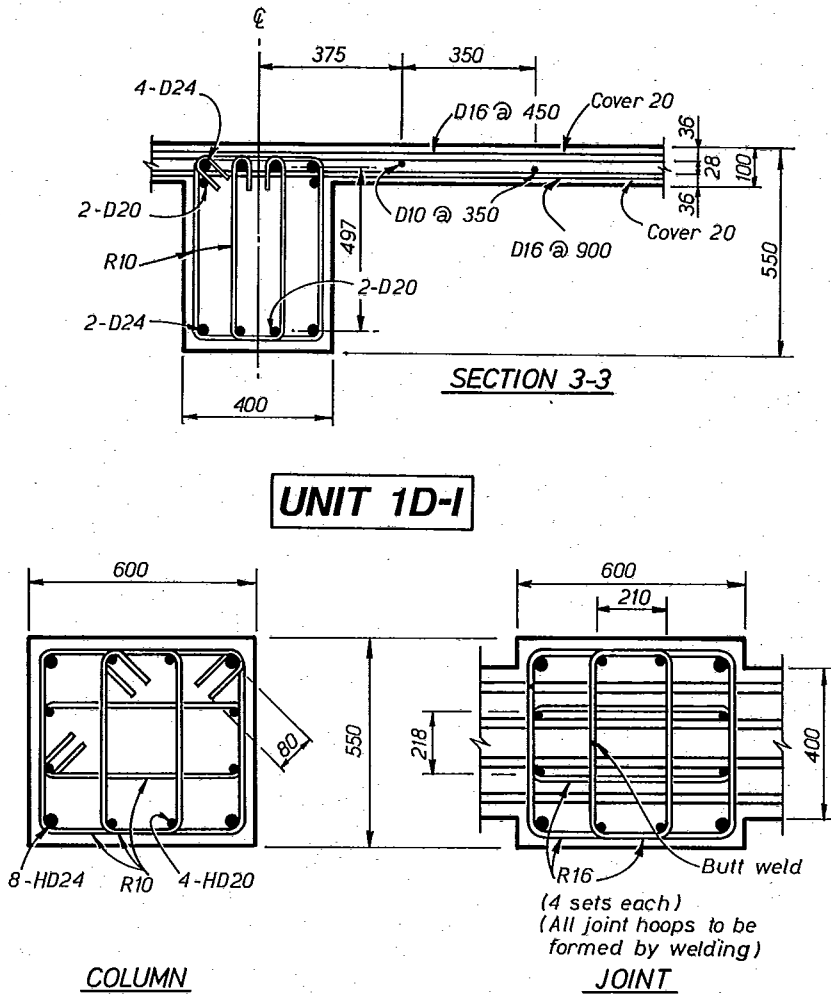
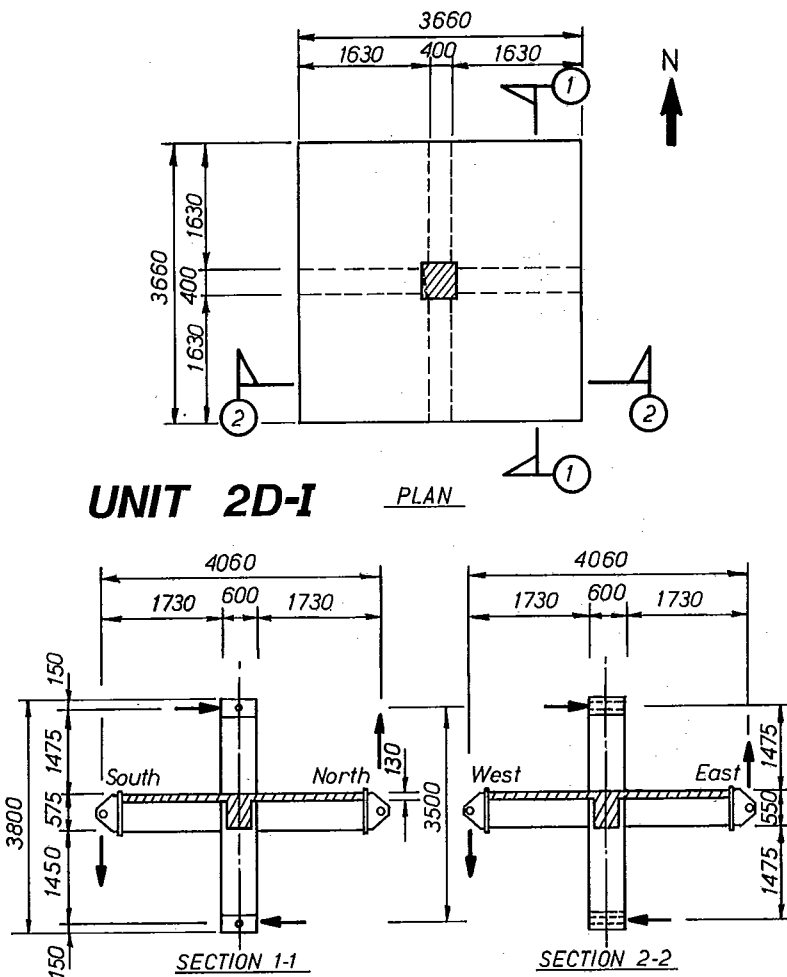


Fig. 1 cont.--Details of Unit ID-I

**NOTES:**

1. Unless otherwise noted concrete cover to main reinforcement to be as follows:
40mm for beams & columns
20mm for slabs
2. Grades of reinforcing to be as follows:
R = Grade 275 round bars
D = Grade 275 deformed bars
HD = Grade 380 deformed bars

Fig. 2--Details of Unit 2D-I

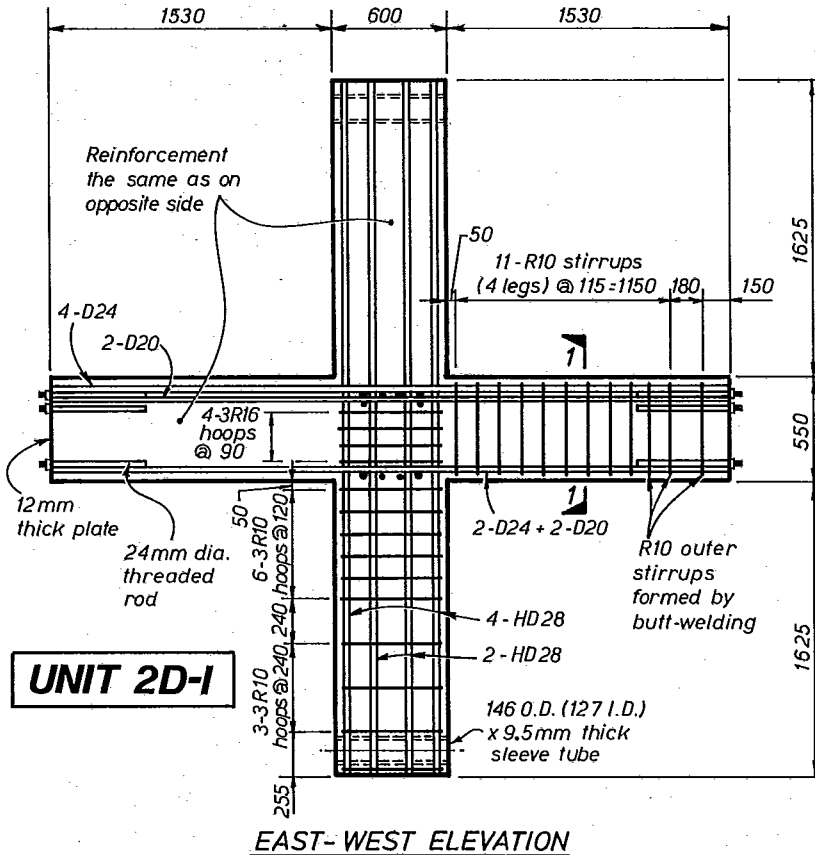
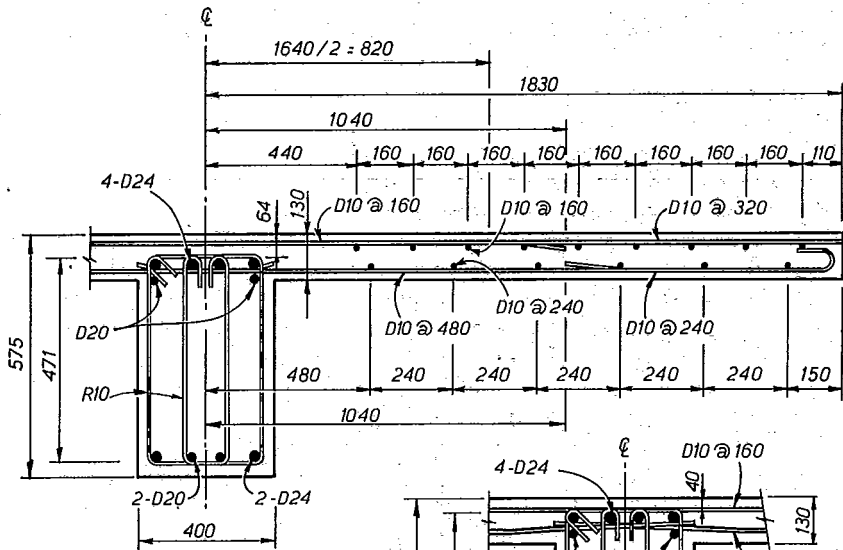
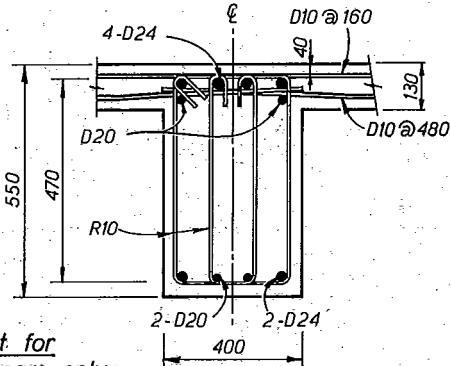


Fig. 2 cont.--Details of Unit 2D-I



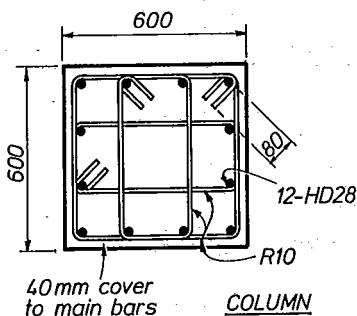
*Reinforcement for
north-south beam & slab*

SECTION 2-2

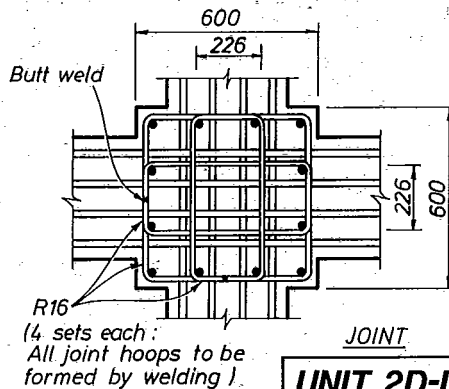


*Reinforcement for
east-west beam only*

SECTION 1-1



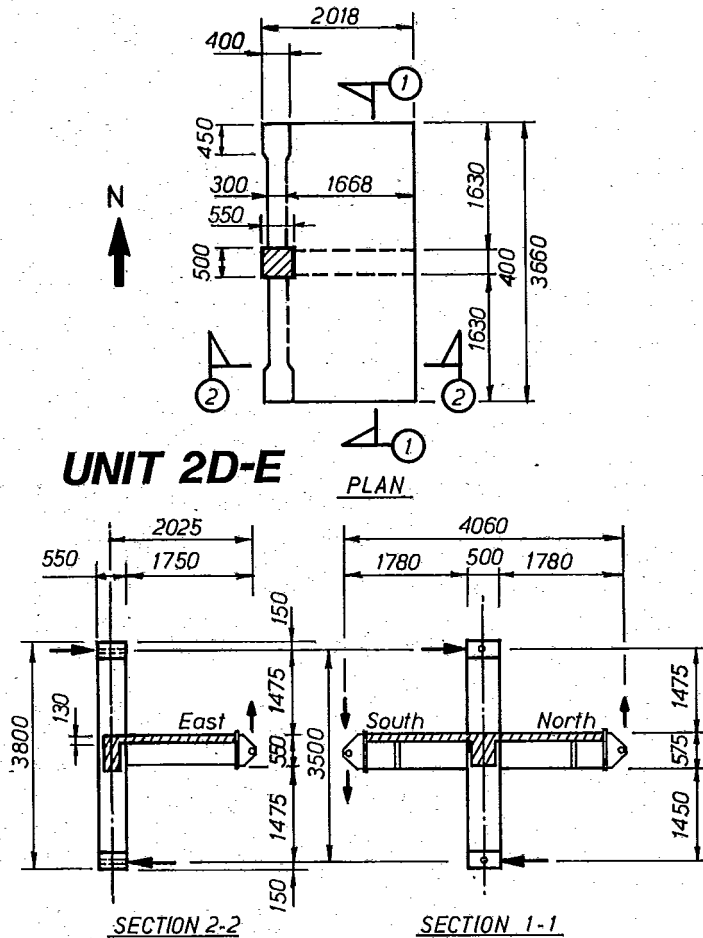
COLUMN



JOINT

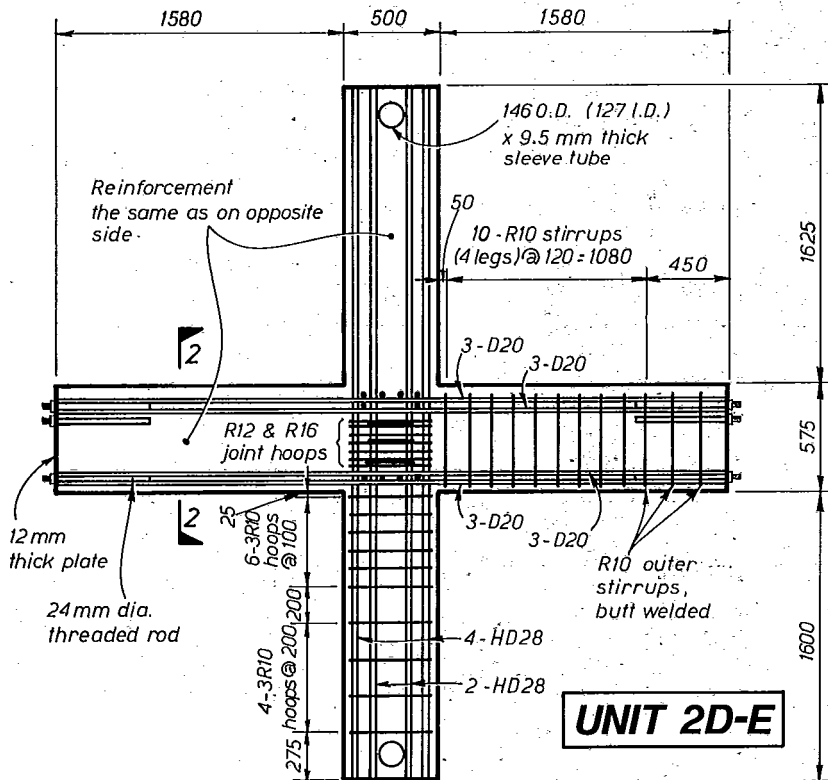
UNIT 2D-I

Fig. 2 cont.--Details of Unit 2D-I

**NOTES:**

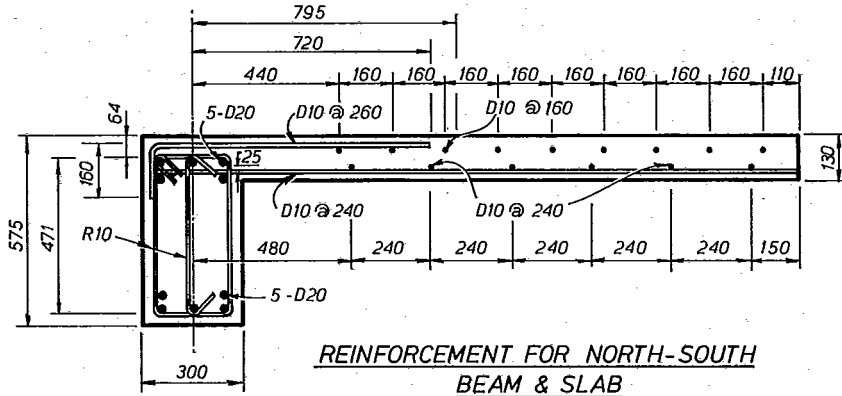
- | | |
|---|--|
| <p>1. Unless otherwise noted concrete cover to main reinforcement to be as follows:</p> <p>40mm for beams & columns</p> <p>20mm for slabs</p> | <p>2. Grade of reinforcing bars to be as follows:</p> <p>R - Grade 275 round bars</p> <p>D - Grade 275 deformed bars</p> <p>HD - Grade 380 deformed bars</p> |
|---|--|

Fig. 3--Details of Unit 2D-E



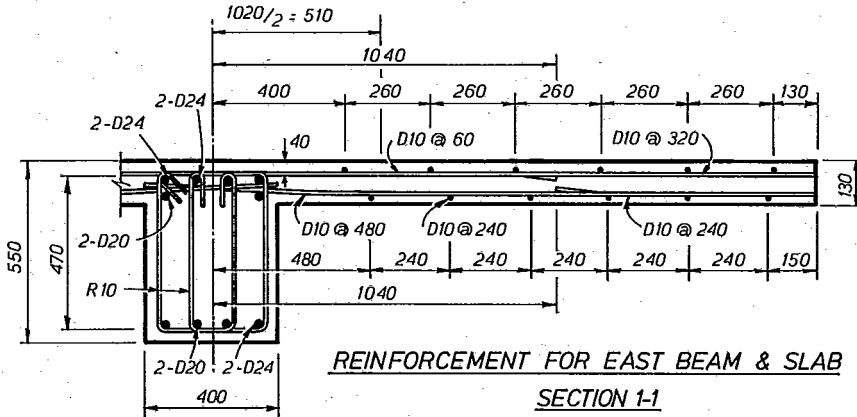
(b) NORTH-SOUTH ELEVATION

Fig. 3 cont.--Details of Unit 2D-E

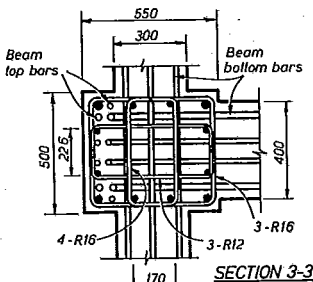


UNIT 2D-E

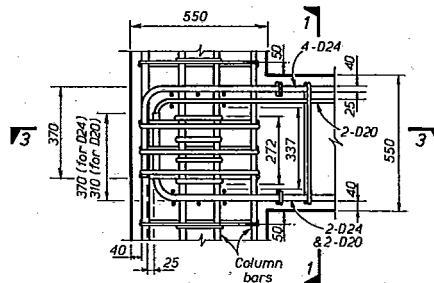
SECTION 2-2



SECTION 1-1

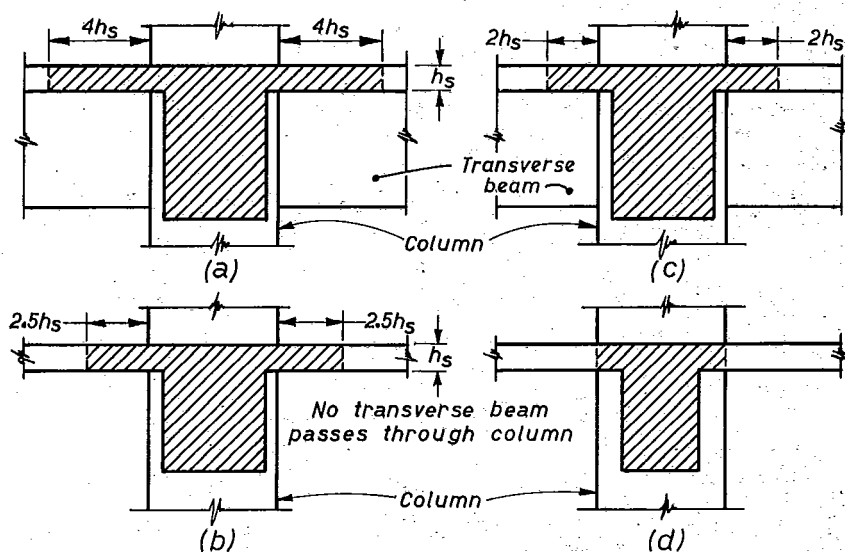


3 sets of R12 & R16 hoops
4 sets of R16 hoops
(all butt-welded)



BEAM-COLUMN JOINT DETAIL

Fig. 3 cont.--Details of Unit 2D-E



At interior columns

At exterior columns

Fig. 4--Effective width of T-beam subjected to negative bending moment, within which the slab flexural reinforcement is considered as being effective in tension

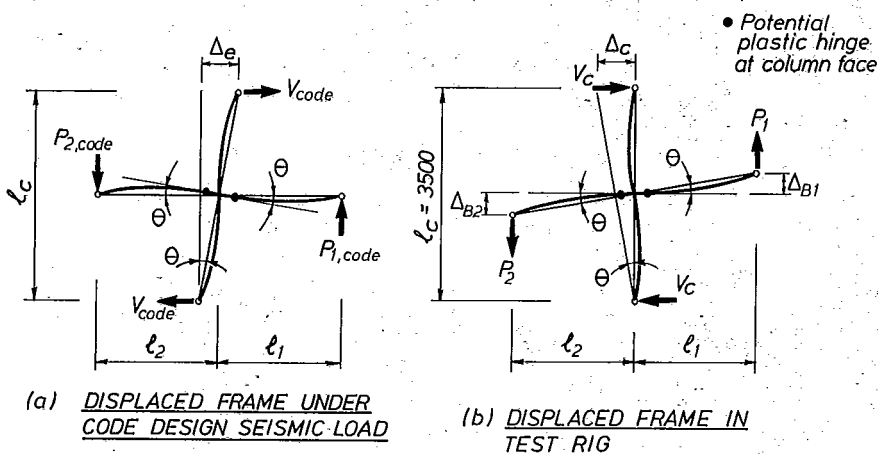


Fig. 5--Determination of interstory displacement and forces for the test units

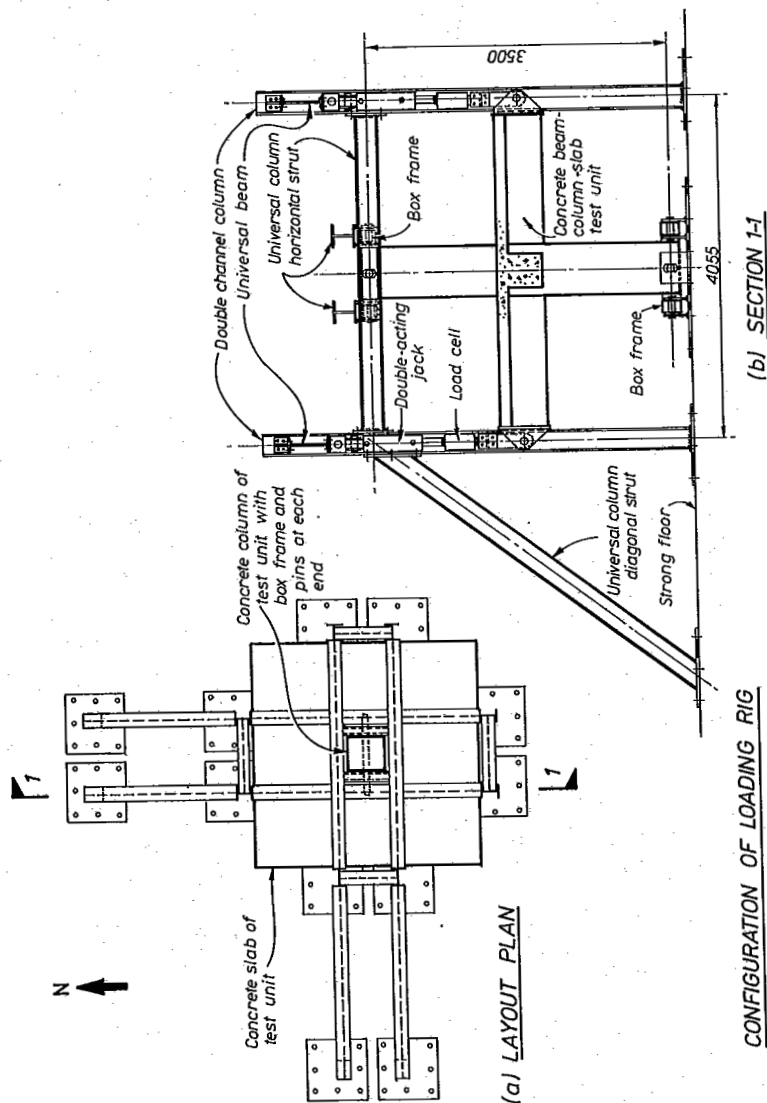


Fig. 6--Views of the loading rig

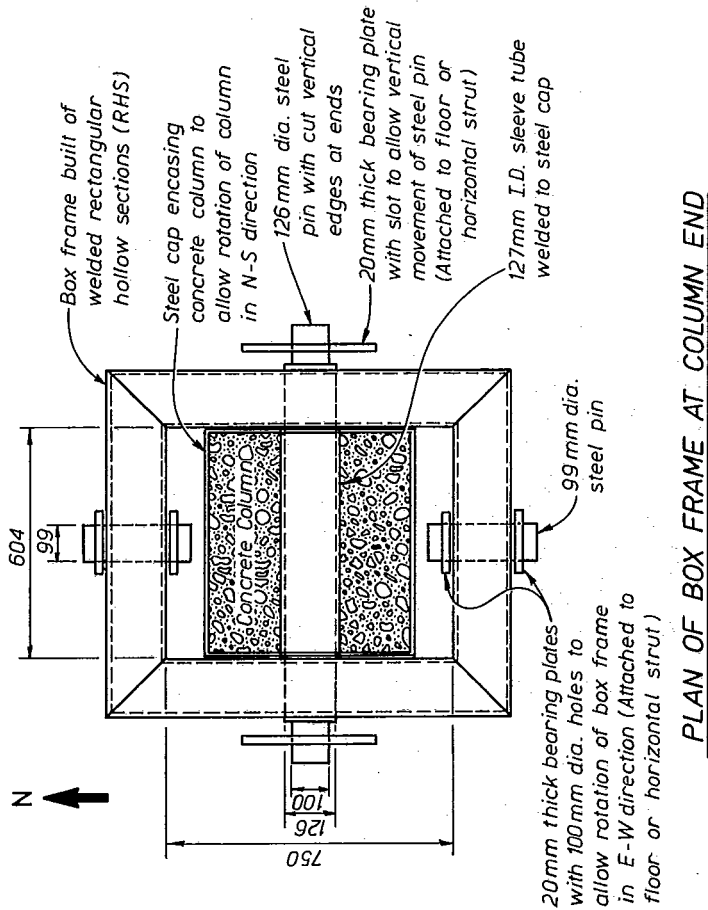


Fig. 6 cont.-- Views of the loading rig

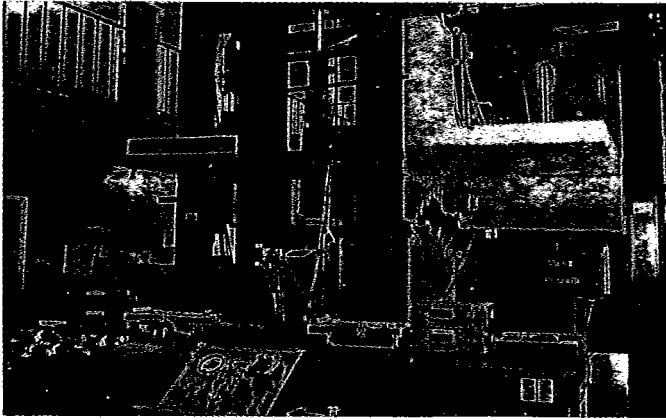


Fig. 7--General view of a unit during testing

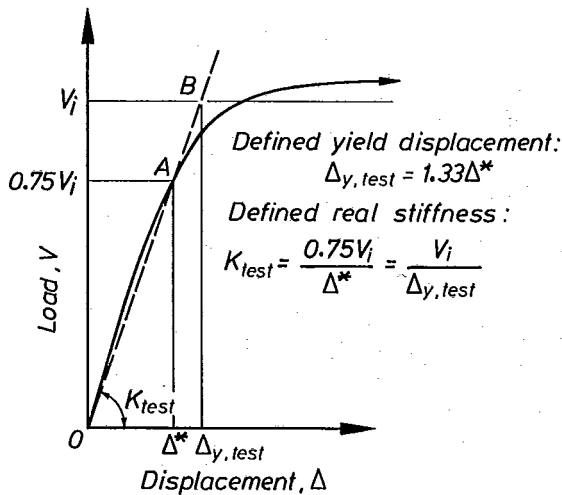


Fig. 8--Determination of first yield displacement and elastic stiffness for the units

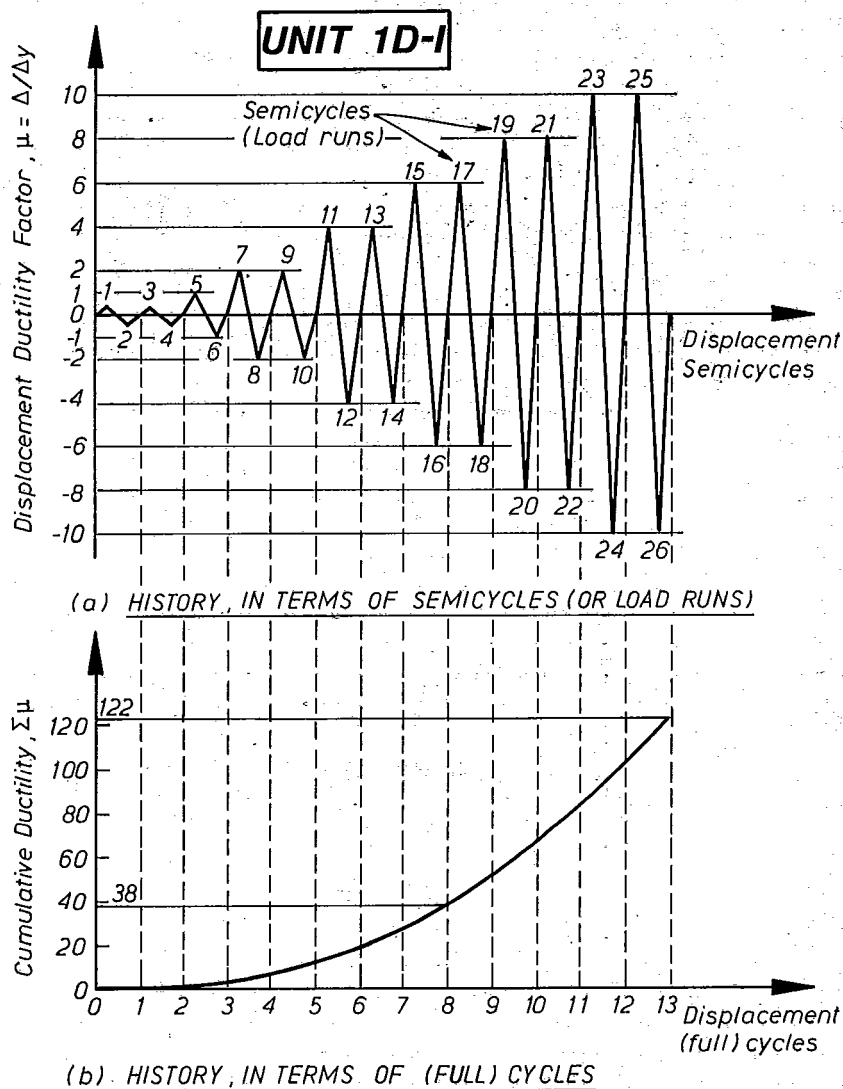


Fig. 9--Quasi-static cyclic loading history for Unit 1D-I

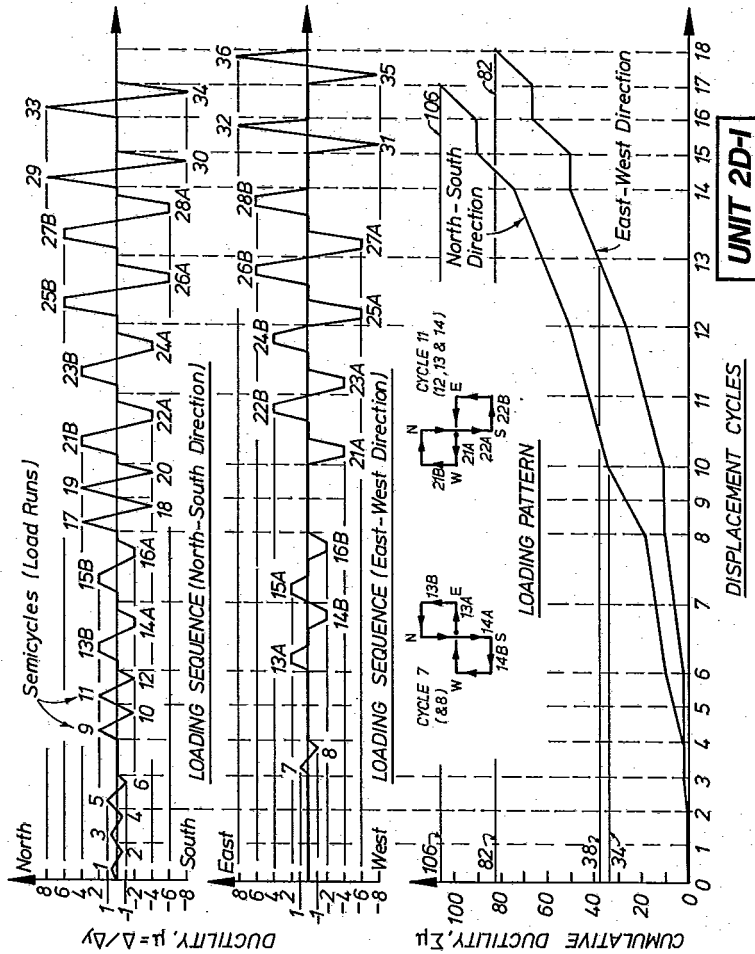


Fig. 10--Quasi-static cyclic loading history for Unit 2D-I

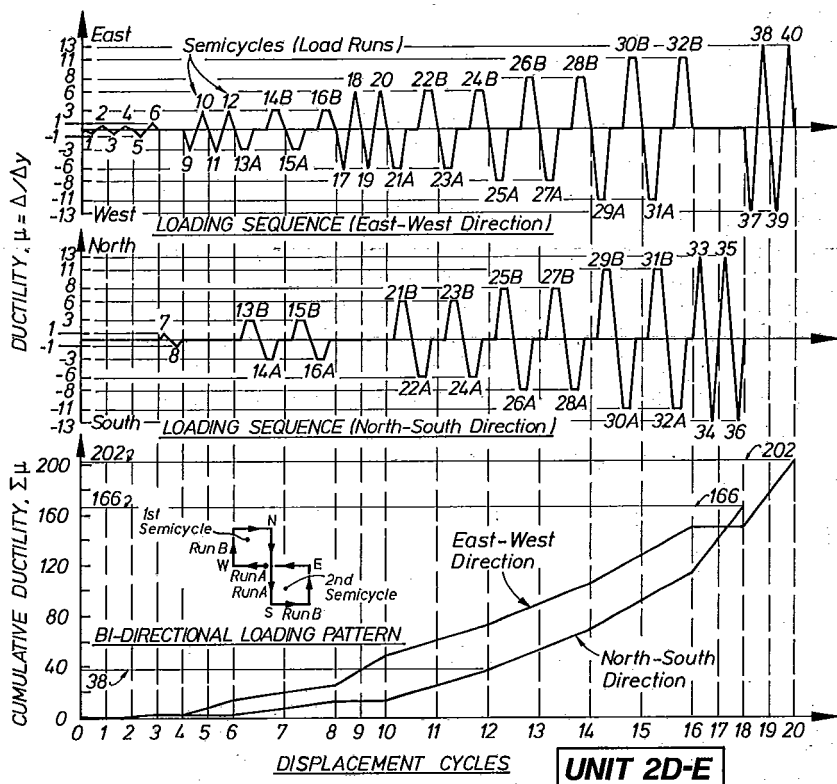


Fig. 11--Quasi-static cyclic loading history for Unit 2D-E

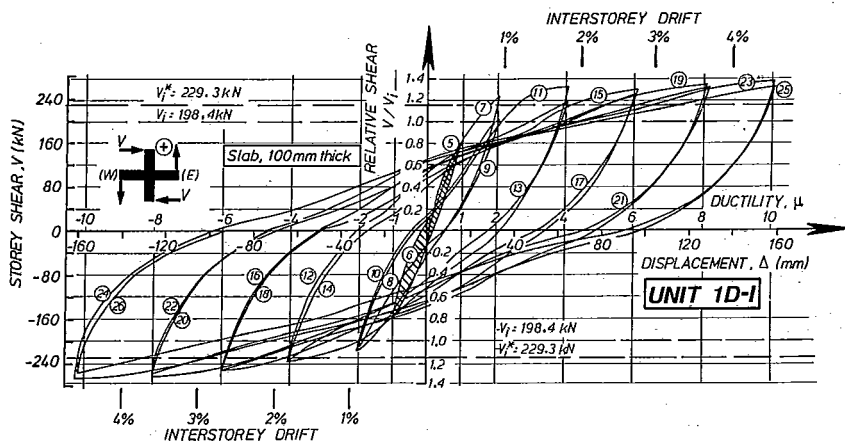
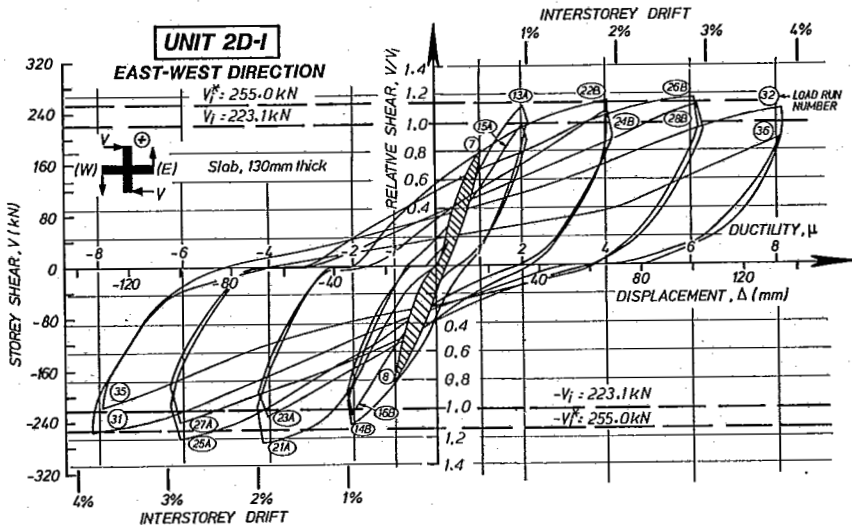
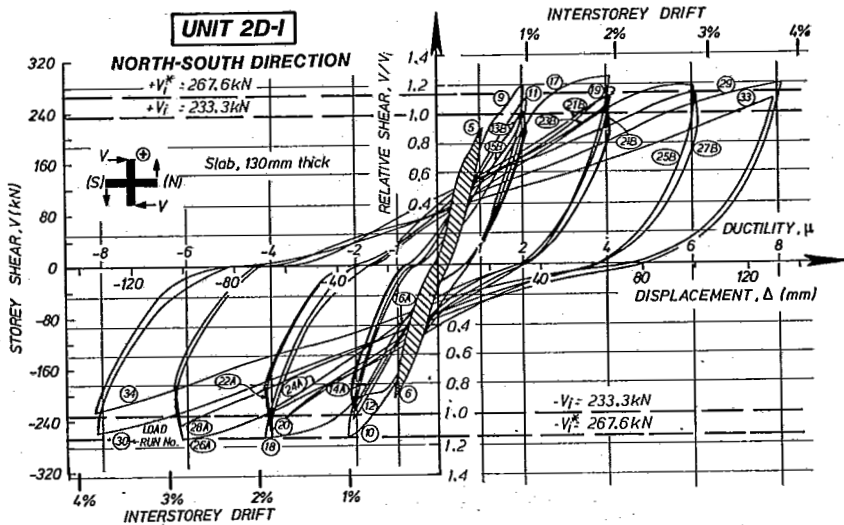


Fig. 12--Lateral load (story shear) versus lateral displacement response of Unit 1D-I

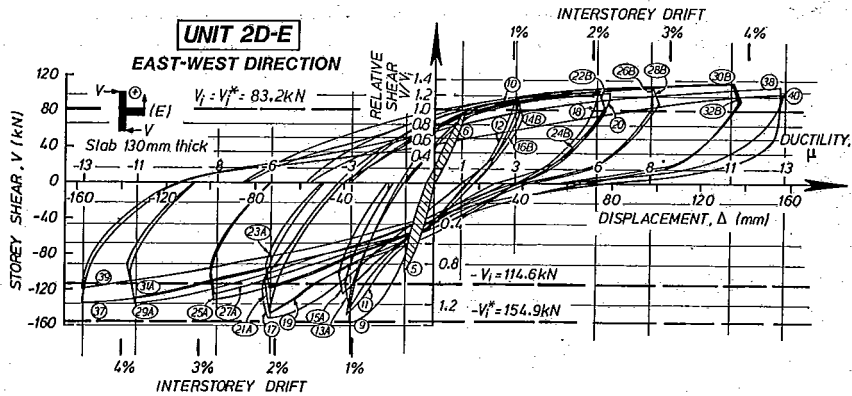


a) East-West direction

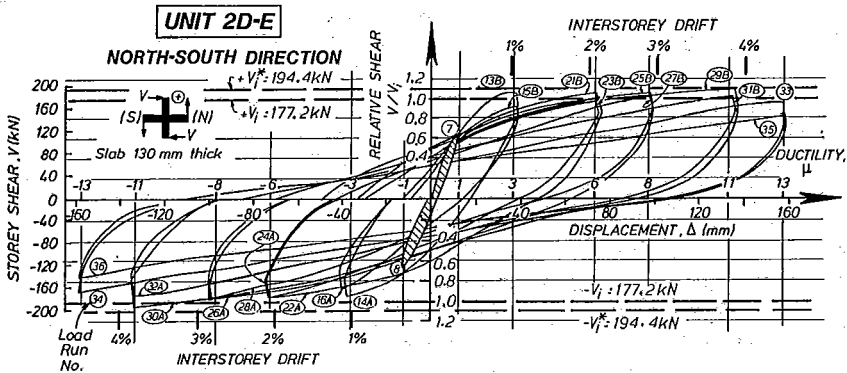


b) North-South direction

Fig. 13--Lateral load (storey shear) versus lateral displacement response of Unit 2D-I



a) East-West direction



b) North-South direction

Fig. 14--Lateral load (story shear) versus lateral displacement response of Unit 2D-E

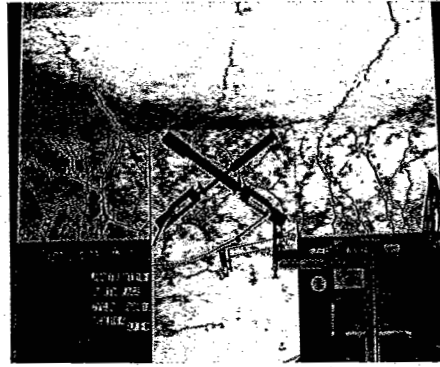


Fig. 15--Unit 1D-I in cycle 9 to ductility of $\mu=6$



Fig. 16--Unit 2D-I in cycle 10 to ductility of $\mu=4$



Fig. 17--Unit 2D-E in cycle 10 to ductility of $\mu=6$

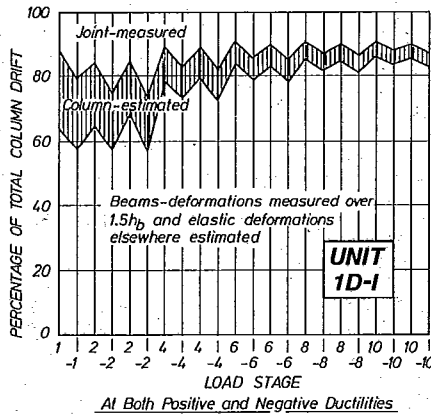


Fig. 18--Contribution to interstory lateral displacements during progressive loading of Unit 1D-I

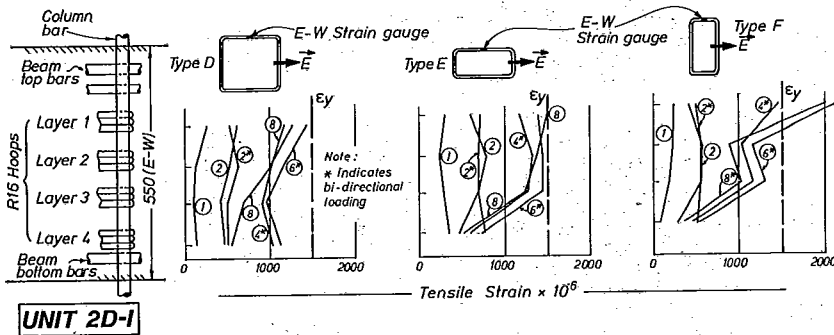


Fig. 19--East-West direction hoop strains during East-West direction loading to positive ductilities of Unit 2D-I

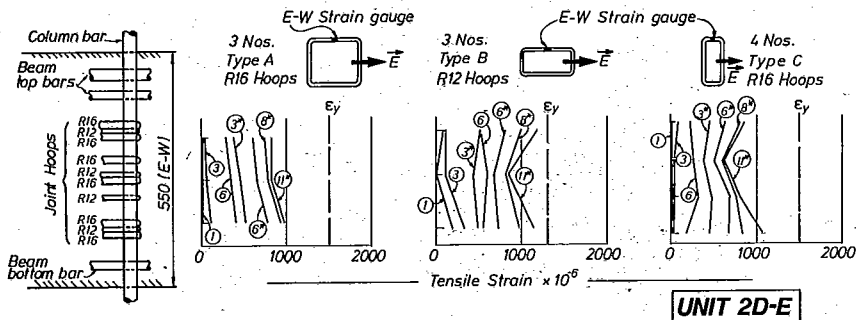


Fig. 20--East-West direction hoop strains during East-West direction loading to positive ductilities of Unit 2D-E

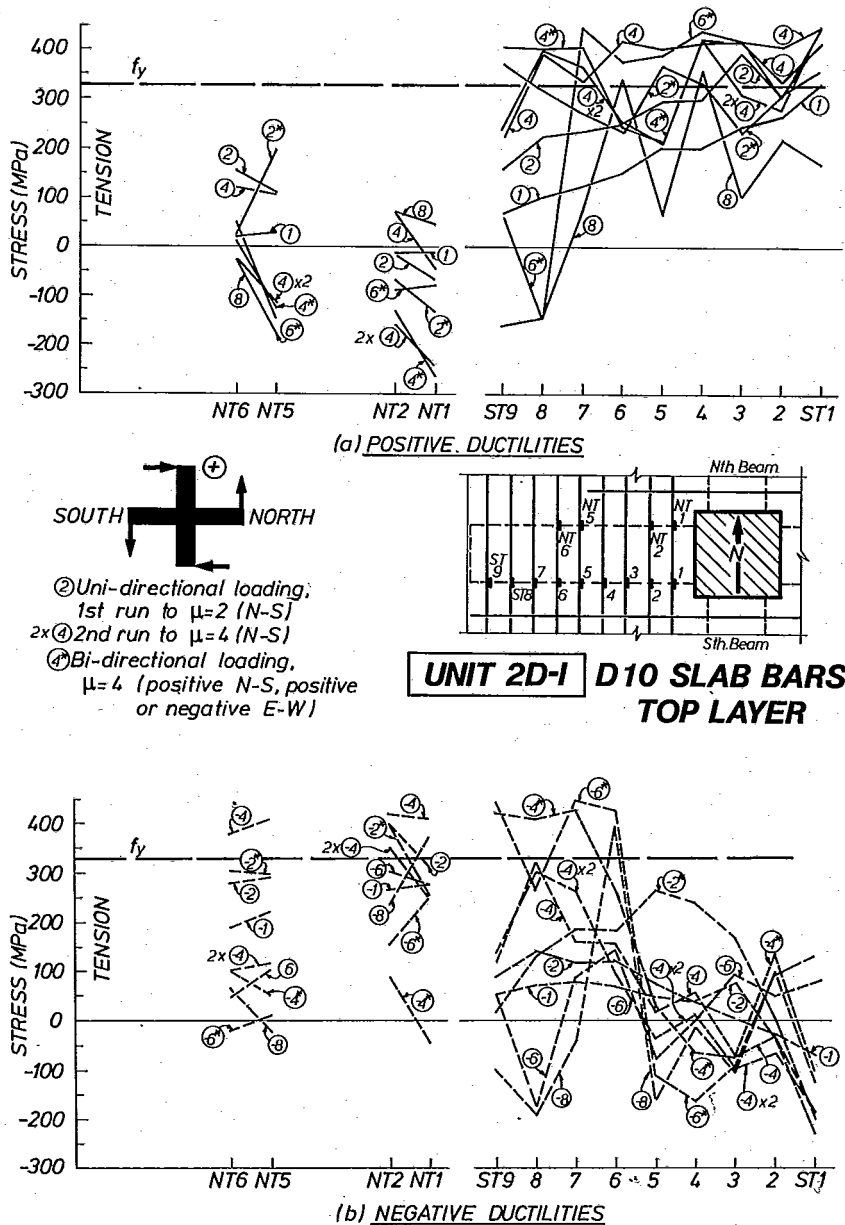


Fig. 21--Stresses in North-South top layer slab bars of Unit 2D-I

SP 123-2

Evaluation of Slab-Beam-Column Connections Subjected to Bidirectional Loading

by Y. Kurose, G.N. Guimaraes,
L. Zuhua, M.E. Kreger, and J.O. Jirsa

Synopsis: Three reinforced concrete beam-column connections with a slab were tested under the U.S. - Japan - New Zealand - China cooperative research program on design of beam-column connections. Two of the specimens were subjected to both unidirectional and bidirectional cyclic loads. Results of the tests are described with the aid of story shear - drift angle relations, story shear orbits, measured joint shear deformations, and plots of drift angle components. Various methods for calculating joint shear strength are evaluated in light of the test results. Test data are also used to determine slab effective widths for use in calculation of uncracked beam stiffness and beam flexural capacity.

Keywords: Beams (supports); columns (supports); connections; cyclic loads; evaluation; reinforced concrete; shear strength; slabs; tests