EXPERIMENTS ON THE SEISMIC RESPONSE

OF A MODEL OF SHEAR WALL WITH FRICTION JOINTS

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A Thesis

in the

Centre for

Building Studies

Faculty of Engineering

ABSTRACT

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Parvaneh-Baktash

This thesis indicates that the use of friction type energy absorbers in vertical connections can play an important role in the seismic response of concrete structures.

Study was done on a model using limited slip bolted joints, excited on a shaking table. By varying the slip load it was shown that there is an optimum value which maximizes the energy dissipation and minimizes the stress for a given seismic intensity.

ACKNOWLEDGEMENTS

ACKNOWLEDGEMENTS

I would like to express my sincere thanks to Professor Cedric Marsh, my supervisor and advisor, for his guidance and help throughout this study.

Additional thanks are due to Mr. J. Zilka, lab technician, who aided me on numerous occasions during the experiments. As well, Sandy Pritchard is acknowledged for the typing of this thesis.

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CHAPTER I

INTRODUCTION

INTRODUCTION

The effects of severe ground motions due to earthquakes are receiving increasing attention in reinforced concrete structures, particularly in high rise buildings.

Energy dissipation is a key factor in the effect of strong motion earthquakes. The energy is fed into the structure during the earthquake, and must be dissipated by the structure. Elastic strain energy is conserved, thus energy can only be dissipated by internal friction or inelastic deformation in structural or nonstructural elements.

In most concrete structures it is uneconomical, and in large panel structures it is almost impossible, to dissipate kinetic energy, caused by severe earthquate excitations, by means of ductility, i.e., there is no ability of the structure to perform past the elastic limit. To solve such a problem it is necessary to develop some kind of mechanism which can efficiently consume energy. There are several types of such mechanism. Some are based on plastic deformation of material, others are based on hydraulic dampers, while the method in this study is based on friction.

Experimental and theoretical works have demonstrated (1, 2, 3, 4) that the installation of energy absorbers enhances the earthquake resistance of structures. Further development of the energy dissipation devices and studies of applications should lead to their broader use in structures and structural components.

1.1 Cast in Place Shear Walls

In multistorey buildings, shear walls can provide the majority of the strength required for lateral loading.

When shear walls are subjected to wind and moderate earthquake loads, they present an elastic behaviour. However, to avoid destruction during large earthquakes, some means of energy dissipation is required.

As a general rule, carefully designed and detailed structures can provide a good response to such large loads (5, 6).

Studies have shown that the addition of flanges will increase the ductility, stiffness and yielding capacity of a shear wall (7, 8). It was shown that: a) the addition of flanges will increase the curvature ductility of the section; b) thicker flanges will not affect the ductility of the section, but will increase the stiffness and yield resistance. However, when shear walls utilize ductility to resist severe earthquakes, permanent damage is inflicted on the structure.

Another way to dissipate energy is the sliding of construction joints between shear walls (6, 9). During cyclic loads large slip occurs with a reduction in shear resistance (9). When the reinforcement crossing the joints yields, failure occurs. The shear stress-shear slip relationship for a construction joint specimen which was subjected to a few cycles of reverse static loading (5) is shown in Fig. 1.1. It can be seen that the existing damage is almost irreparable.

Sliding across plastic hinge zones can also provide
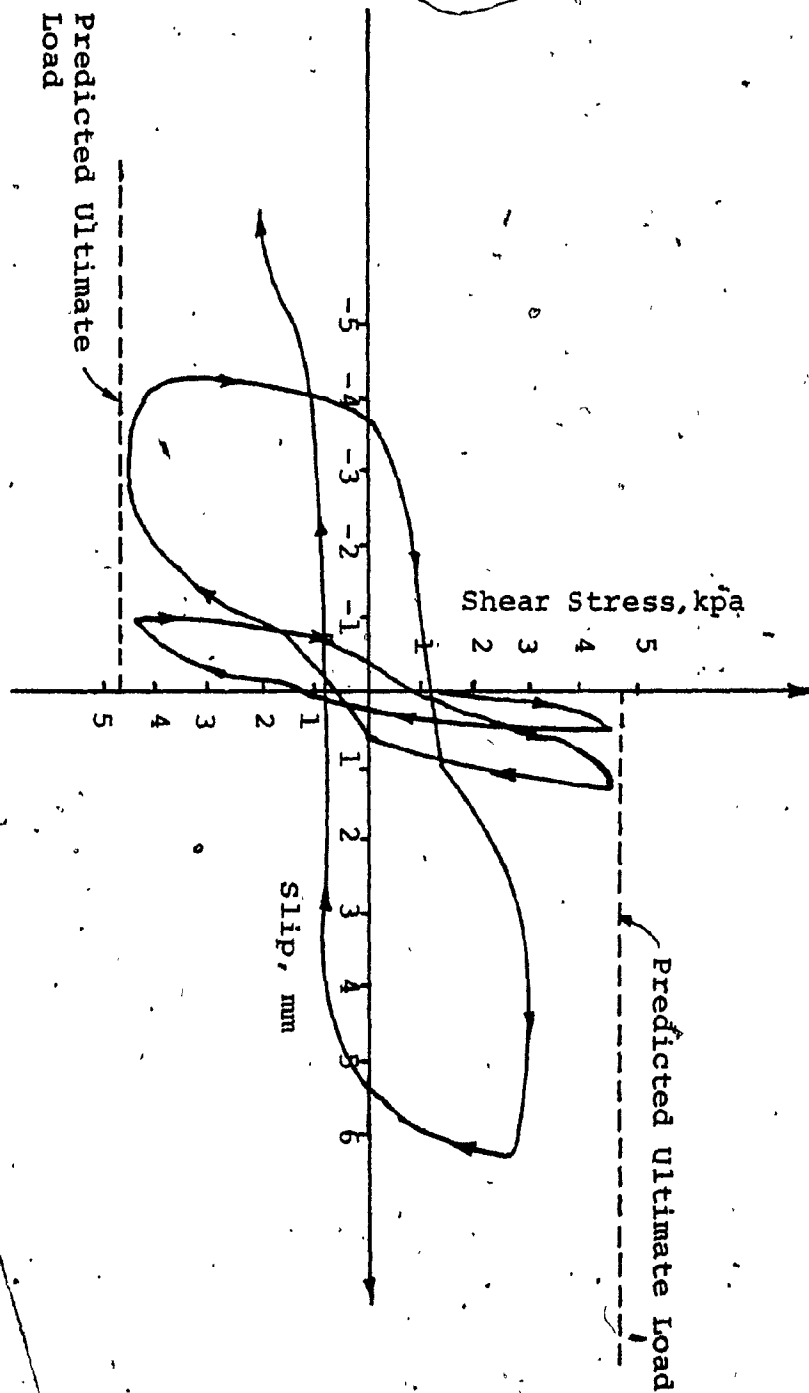
Fig. 1.1 - SHEAR STRESS - SLIP RELATIONSHIP AT A
CONSTRUCTION JOINT TRANSFERRING CYCLIC
REVERSED SHEAR FORCES (5)

large continuous cracks, which usually occur immediately above
the foundation level, where the flexural reinforcement in both
faces yields. This concept led to the design of soft-storey
structures (10). Additional study has shown, elastic super-
structures demand ideal elasto-plastic first storeys, which
can cause early P-Δ collapse (11).

To improve the performance of earthquake resistant
structures, damage has to be minimized and the energy dissi-
pation maximized. Plastic deformation and permanent damage
can be avoided by introducing the limited slip bolted joints
in cast in place solid shear walls or even in the coupling
systems proposed by Gosh and Paulay (7, 8). This kind of
joint must be inserted before pouring the concrete.

1.2 Precast Panel Wall Systems

Precast panel wall systems are widely used on construc-
tion due to simple and fast installation procedures.

Different structural wall systems are defined by the
layout of the load bearing walls. There are three basic
systems: cross wall, long wall, and mixed wall. Their char-
acteristic configurations are shown in Fig. 1.2.

Parallel and perpendicular walls can be joined verti-
cally to provide a large moment of inertia or flange action
(5).

The joints are the most appropriate location for con-
suming energy. There are three joint locations available.
These are shown in Fig. 1.3. In slab to slab joints, the
forces are small, and the energy dissipation through these

One-Way
Floor System

Non-Load Bearing
Long-Wall

Load Bearing
Cross-Wall

CROSS-WALL
SYSTEM

One-Way
Floor
System

Non-Load Bearing
Cross-Wall

Load Bearing
Long -Wall

LONG-WALL
SYSTEM

Two-Way
Floor
System

Load Bearing
Walls

TWO-WAY
SYSTEM

Fig. 1.2 - PRECAST PANEL WALL SYSTEMS

vertical
wall-to-wall
joint

slab-to-slab joint

horizontal wall-to-wall joint
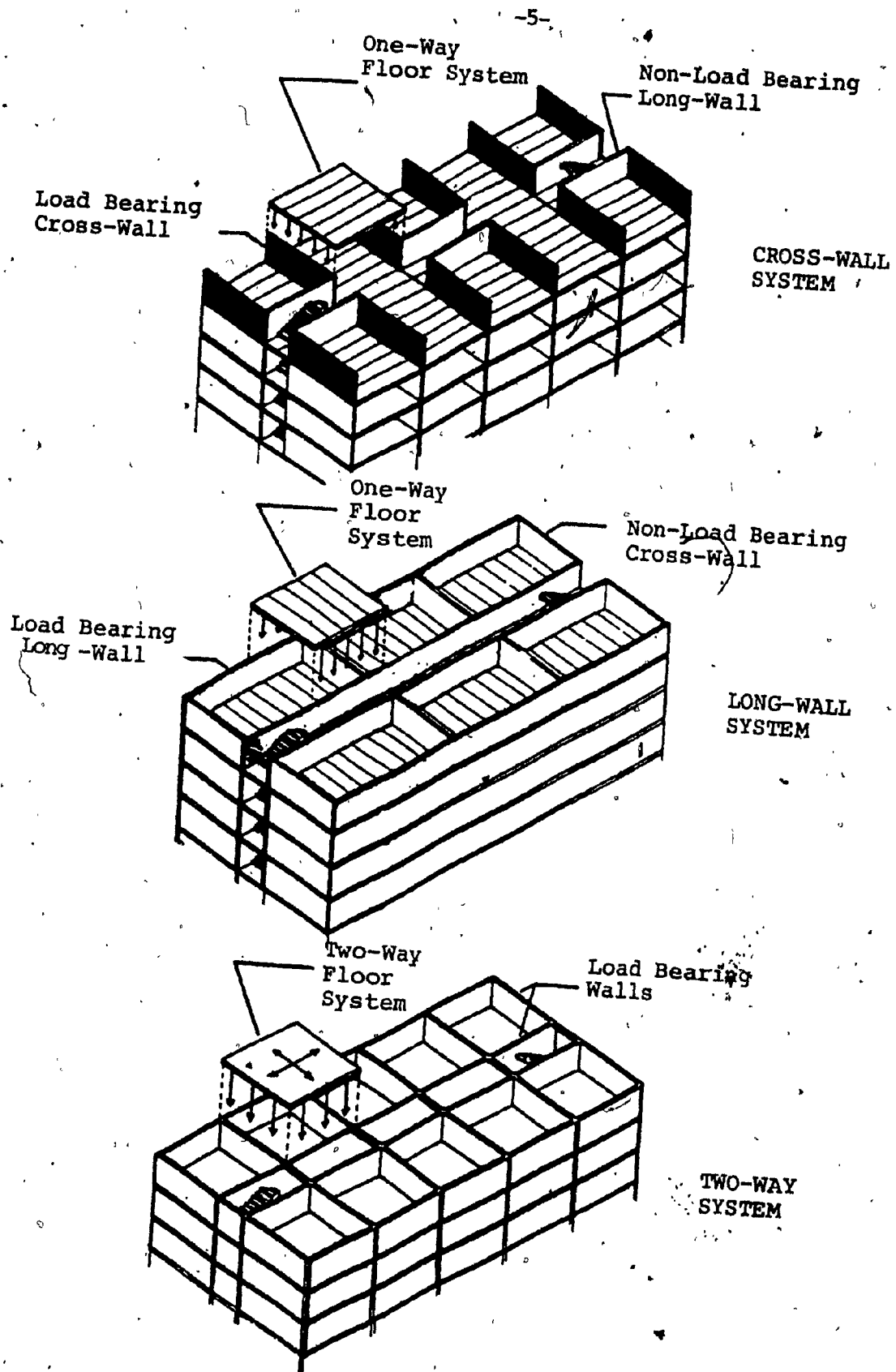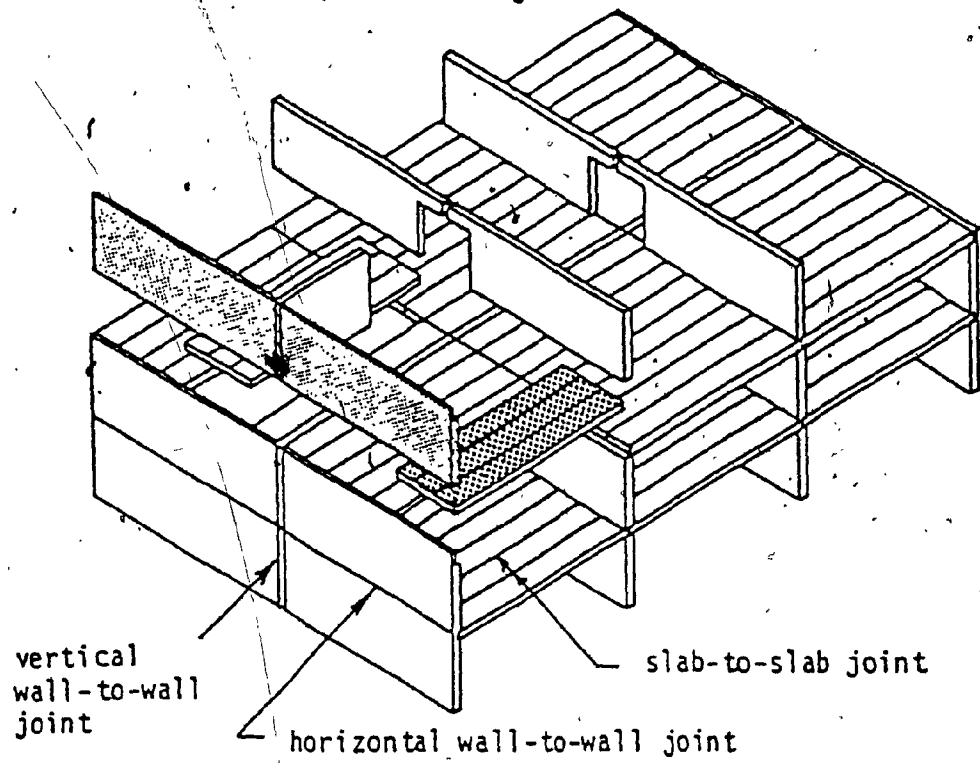
Fig. 1.3 - TYPES OF JOINTS IN LARGE PANEL BUILDING

a) NO SHEAR SLIP   b) HORIZONTAL SHEAR SLIP   c) ROCKING d) VERTICAL SHEAR SLIP
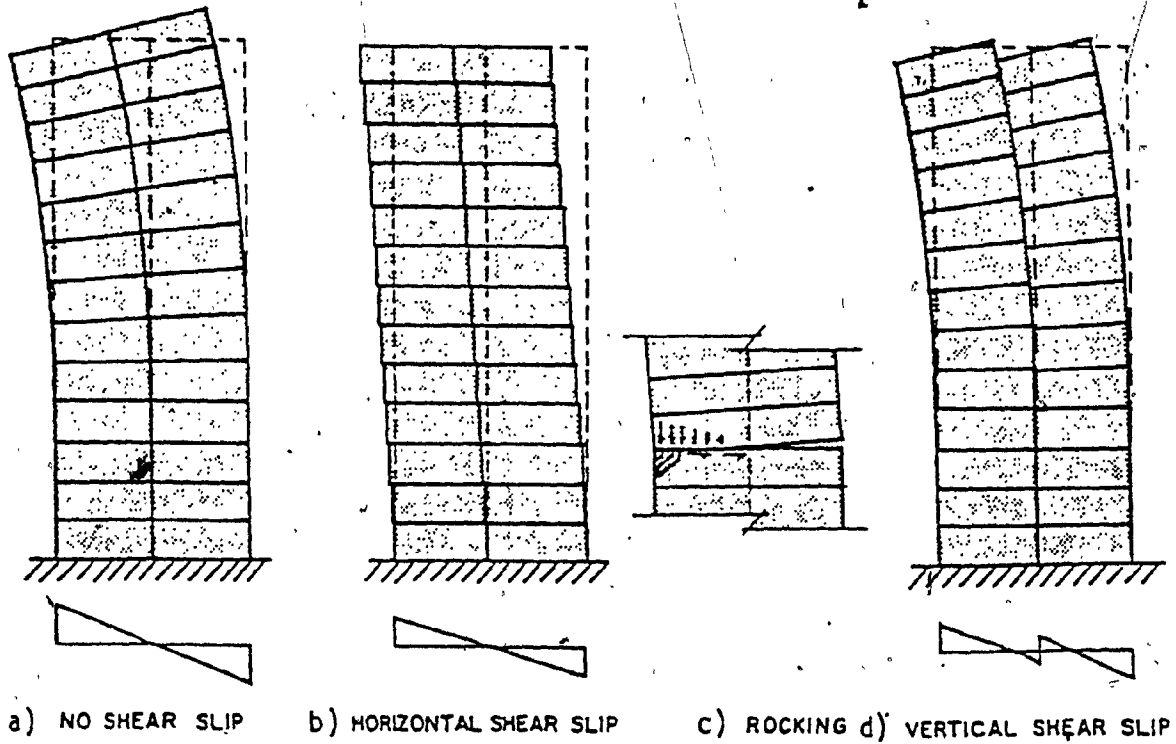
Fig. 1.4 - MODES OF DEFORMATION

a) Plain   b) Grooved   c) Castellated (Keyed)
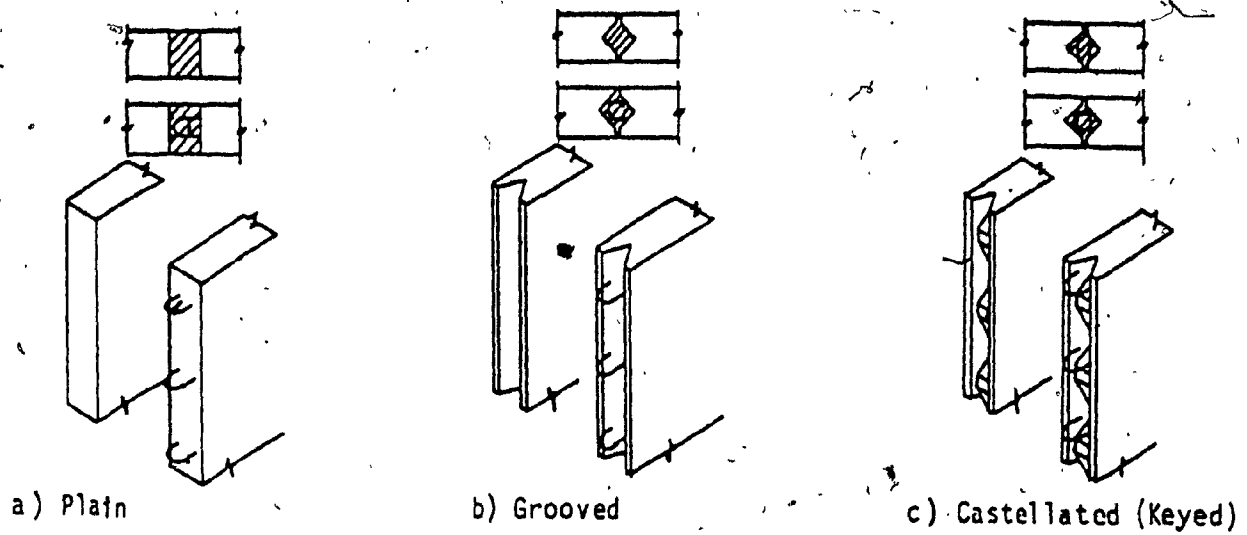
Fig. 1.5- DETAILS OF WET VERTICAL JOINTS [13]

loops
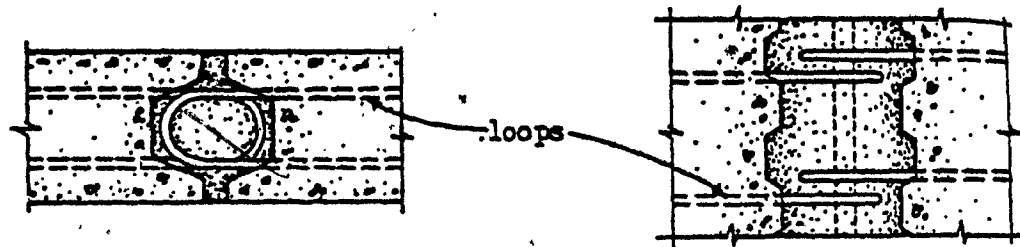
Fig. 1.6 - VERTICAL LOOP JOINT WITH KEYS, WET CONNECTION

joints cannot be considered. Horizontal joints are permanently
damaged if they slip, with the destruction of the interfaces
(Fig. 1.4b). They also may receive alternating tension and
compression, causing opening and closing of the joint, pro-
viding a rocking motion (Fig. 1.4c). Vertical wall to wall
joints have been shown to be the most suitable place for
energy dissipation (1, 2, 12) (Fig. 1.4c).

Vertical connections may be either "wet" or "dry".
Wet joints are made with poured in place concrete or grout,
to transfer stresses at the connections, and can be reinforced
to resist the tensile stresses while the compressive stresses
are resisted by concrete (13) (Fig. 1.5). The keyed joints
are the most common ones (Fig. 1.5c). Band breakage with
subsequent dislocation of the keys is a customary failure
mode of unreinforced keyed joints. There are essentially
three ways of reinforced keyed joints to fail: yielding,
diagonal tension, shearing of the keys or crushing of the
keys. The joints can present reasonable ductility after
yielding by the use of steel continuity, which can be pro-
vided by lapped dowels, lapped loops with longitudinal pins,
mechanical splices or welds (Fig. 1.6).

In dry connections, the connecting pieces are embedded
in the panel during the casting process, and then joined to
the adjacent panel in the field. Connectors are either welded
or bolted together. Three possible ways of dry connection
failure are: concrete cracking, metal failure, or insert
pullout. Concrete may fail due to diagonal tension.

Concrete also may fail in compression under the studs, with crushing and spalling of the concrete. Inserts of reinforcement may pull out due to loss of bond.

All the above failure modes can be avoided by the use of bolted, dry joints, provided with slots to allow slipping. This claim is based on a recent study by Pall et al (1, 2) which shows the potential of such a solution.

1.3 Proposed Mechanism

The importance of developing a good mechanism for consuming energy during earthquakes without producing permanent damage to the structure, has been stressed in recent studies (10, 11, 12, 14).

The Limited Slip Bolted (LSB) joint is one of the mechanisms that has shown a reasonable hysteretic characteristic for a number of reversals of loading. It consists of steel plates with slotted holes clamped together with high tension bolts, dissipating energy through friction when the plates slide. Figs. 1.7 to 1.9 show the details. The force at which the joints start to slip depends on the coefficient of friction and the clamping force of the bolts.

During wind and moderate earthquakes the joints must act as rigid, producing a monolithic wall, thus transfering the full shear force.

During severe earthquakes the joints start to slip with near "elasto-plastic" behaviour, while the shear walls stay in their elastic range.

a) Load-deformation curve   b) hysteresis loop
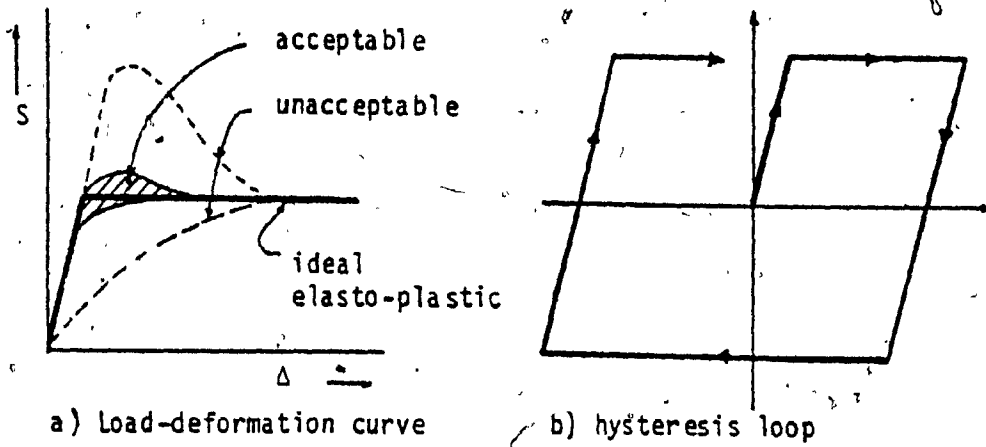
Fig. 1.7 - IDEAL BEHAVIOUR OF VERTICAL JOINT
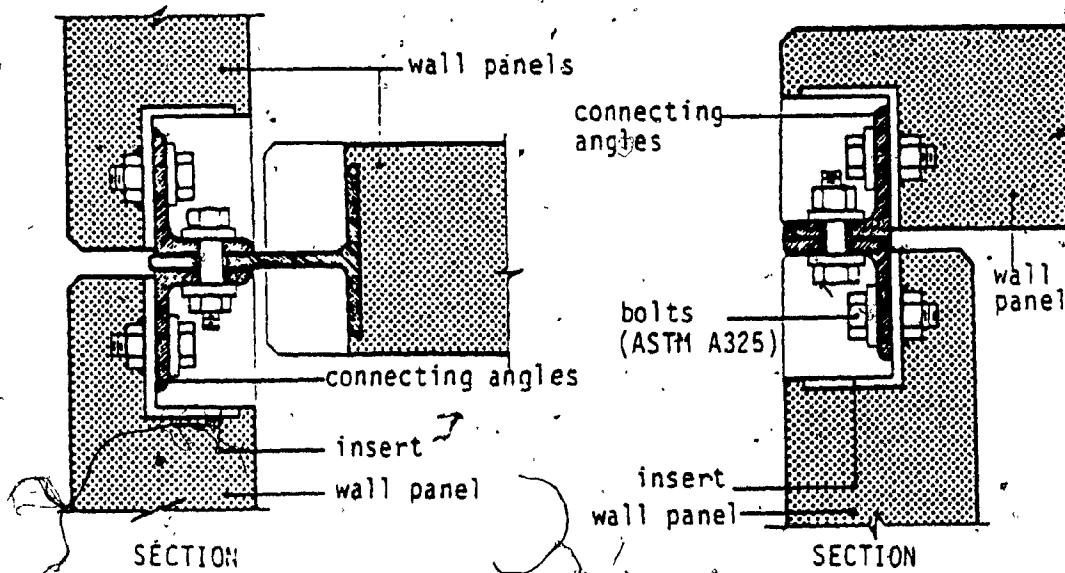
Fig. 1.8 - CORNER WALL - TO - WALLJOINT

weld after erection

connecting plate

rect. washers

insert

joint

ELEVATION

INSERT

anchors

CONNECTING PLATE

connecting plate

bolts (ASTM A325)

insert

panel

nuts welded to insert
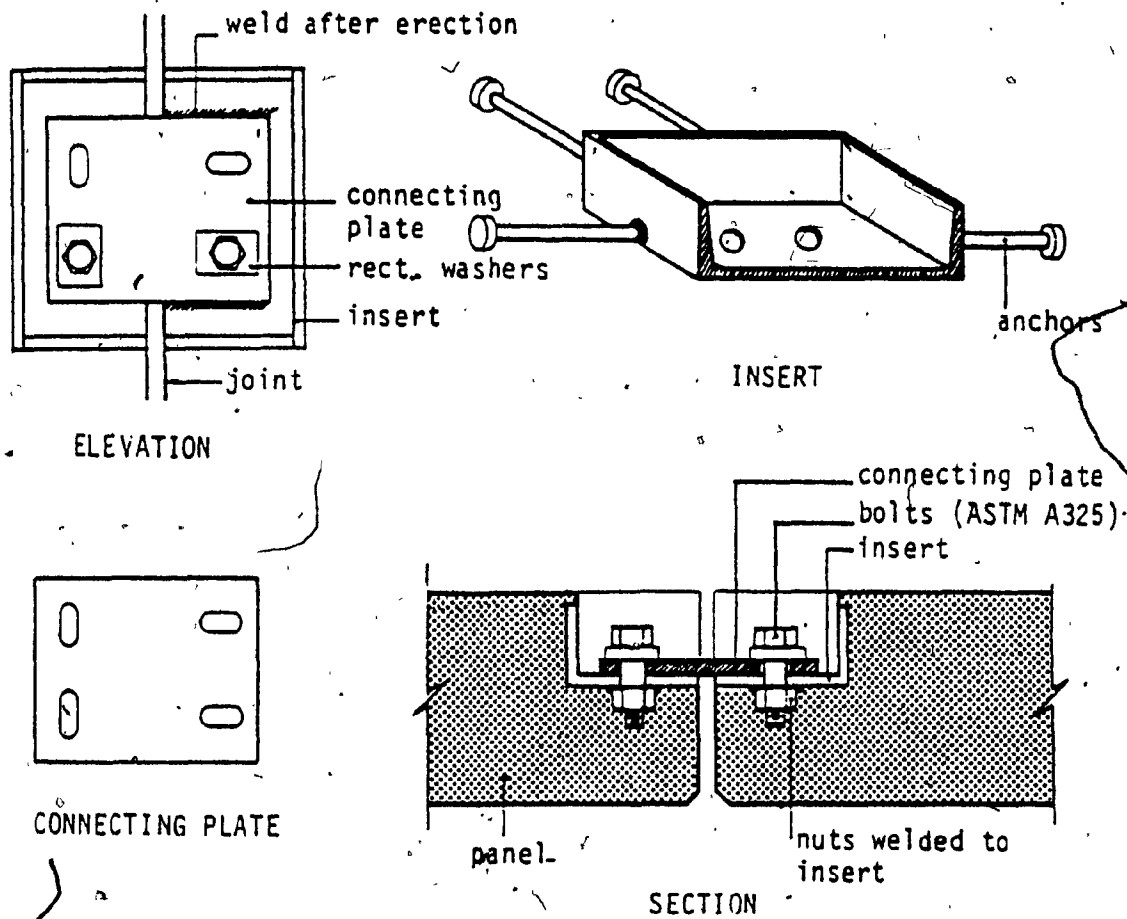
SECTION

Fig. 1.9 -WALL - TO -WALL JOINT

Static and dynamic cyclic tests were conducted on several types of connection (Fig. 1.10), studying different laying surface treatments to evaluate their basic design properties. The best behaviour was shown by heavy brake lining pads inserted between steel plates with mill scale surfaces.

1.4 Object of the Present Project

In theory, the response of structures to known exciting forces can be solved. However, to have any practical value, the basic dynamic properties require many details of material behaviour and structural configurations that cannot be fully analyzed. Therefore direct experimental confirmation is desirable.

Thus, the object of the present study was to find the optimum slip force, for shear walls coupled with LSB joints, by use of a model on a shaking table, and compare it with theoretical values. The energy dissipation by these joints is a function of the slip load and the length of travel. The optimum solution will depend on the rigidity and mass of the building and the seismic intensity.

The mass and rigidity of the model were held constant and the slip load varied to determine the optimum value for a given ground acceleration.

1.5 Organization of the Text

The thesis has been arranged in the following manner:

In Chapter II a detailed description of the dynamic tests and the experimental set-up are described.
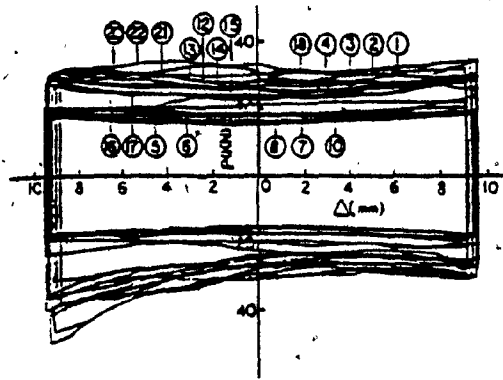
a) Mill Scale
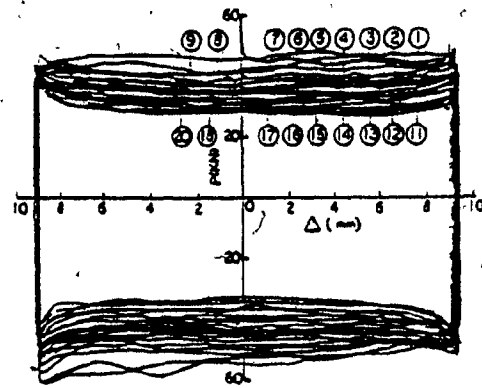
b) Sand Blasted
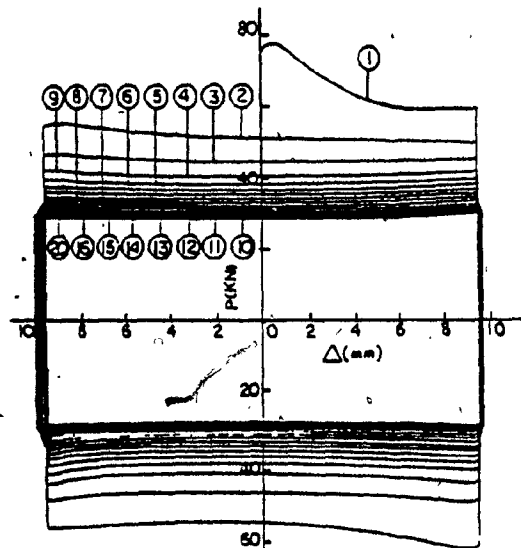
c) Inorganic Zinc-Rich Paint
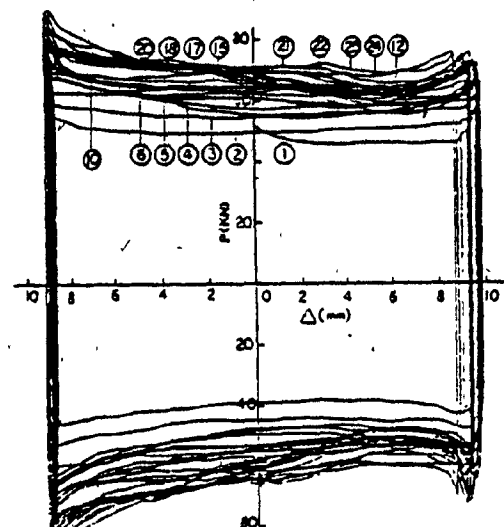
d) Metalized
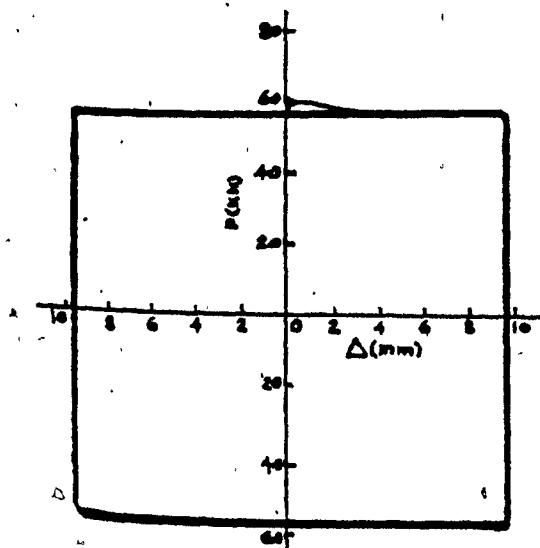
e) Brake Lining Pads
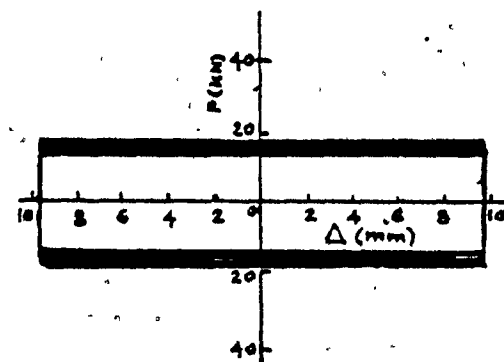
f) Polyethylene Coating

Fig. 1.10 - HYSTERSIS LOOPS OF LIMITED SLIP BOLTED (LSB) JOINTS

Chapter III deals with the test procedure and the obtained results.

Chapter IV is directed towards the methods of analysis and the proposed procedure for parametric studies with a brief discussion on approximate analysis. The results are compared with those obtained in the tests.

Chapter V summarizes the conclusion of the investigation. Recommendations for future studies are also made.

CHAPTER II

DYNAMIC TESTS AND

EXPERIMENTAL SET-UP

2.1 Dynamic Tests

There are three main types of dynamic tests appropriate for experimental determinations: 1) free vibration tests, 2) shaking table tests, 3) forced vibration tests. These are summarized in the following outline.

2.1.1 Free Vibration Tests

Free vibration tests can be set up by either initial velocity or pull back tests.

2.1.1.1 Initial Velocity Tests

Impact forces can produce initial velocity in structures. This can be generated by falling weights or by a swinging rod with a weighted end, i.e., pendulum. Small rockets (15) or explosive cartridges (16) can also provide impulsive loads. In these tests the time duration for applied force is much shorter than the natural period of the structure.

2.1.1.2 Pull Back Tests

The most common test for producing free vibration in the structure is the pull back test. This can be done by connecting a cable to the top of the structure. After pulling the cable, a sudden release will cause free vibration in the structure about its static equilibrium position. The natural frequency and the damping coefficient can be found from the recorded response.

2.1.2 Shaking Table Tests

A shaking table provides a base, for the model structure, which can produce motions similar to earthquakes.

It is not possible to use these tables for fullscale structures, but they can provide a good response for model tests and structural components. This is the type of test used in the present study.

2.1.3 Forced Vibration Tests

Forced vibration tests on a complete building give the most accurate information.

Forced vibration tests can be divided into three main types: 1) resonance tests, steady state sinusoidal excitation; 2) variable frequency sinusoidal excitation; 3) transient excitations.

The most useful type is the steady-state sinusoidal excitation.

2.2 Experimental Set-Up

2.2.1 Description of the Model

The model used in this study was a stapled assembly of sandwich panels with aluminium extrusions incorporating LSB joints.

2.2.1.1 Panels

The sandwich panels used were made up of a styrofoam core 2in. (50.8mm) thick, with a frame of 2 x 2 in. (50.8 x 50.8mm) clear white pine. The two faces of each panel were .025 in. (.635mm) thick utility aluminium, glued with epoxy on the foam core and wood frame.

The wood and the aluminium facing were tested according to ASTM D143, E8, E238 standards (17). The results are shown in Tables 2.1 and 2.2.

White Pine	Modulus of elasticity = E	Proportional Limit = F	Ultimate Stress = F_u
Compression Parallel to the grain	1000 ksi (7000 MPa)	45 ksi (31 MPa)	6.5 ksi (45 MPa)
Compression Perpendicular to the grain	90 ksi (620 MPa)	.8 ksi (5.5 MPa)	-

TABLE 2.1 - PROPERTIES OF THE WOOD USED FOR SANDWICH PANELS (17)

Aluminum Faces	Yield Stress F_y	Ultimate Stress F_s	Modulus Of Elasticity (E)
Tension	22 ksi (152 MPa)	25 ksi (172 MPa)	10^4 ksi (70000 MPa)
Shear	12 ksi (83 MPa)	15 ksi (103 MPa)	-
Bearing	40 ksi (276 MPa)	-	-

TABLE 2.2 PROPERTIES OF THE ALUMINIUM USED FOR SANDWICH PANELS (17)

2.2.1.2 Extrusions

Two types of extrusion were used, "I" type cross section and hollow square cross section with eight flanges, two at each corner and perpendicular to each other. The dimensions of extrusions are shown in Figs. 2.1 and 2.2. The aluminum used for the extrusions was 6063-T5 produced by Alcan with the following properties (17):

Ultimate tensile stress $\qquad$ F_u= 22 ksi (152 MPa)

Tensile yield stress $\qquad$ F_y= 16 ksi (110 MPa)

Bearing stress $\qquad$ F_b= 46 ksi (320 MPa)

The panels were connected to the extrusions with staples

2.2.1.3 Staples

Staples used in the model were U-shaped, galvanized, made from high strength carbon steel. Their ultimate shear strength according to ASTM B565 standard was 75 ksi (520 MPa) (17). Their dimensions are shown in Fig. 2.3.

The tests on the strength of staples (17) show that increasing the number of staples in a row decreases the slope of the load per staple leg versus the deformation, while the ultimate capacity remains almost the same (Fig. 2.4), so the maximum load that was considered in calculations was 40 lb. (178 N) per staple leg.

Panels were connected to extrusions by using a pneumatic Senco stapler gun (Fig. 2.5), connected to 100 psi (700 KPa) pressure line.

The extrusions were approximately .08 to .1 in.(2.0 to 2.4 mm) wider than the thickness of the panels. A plywood
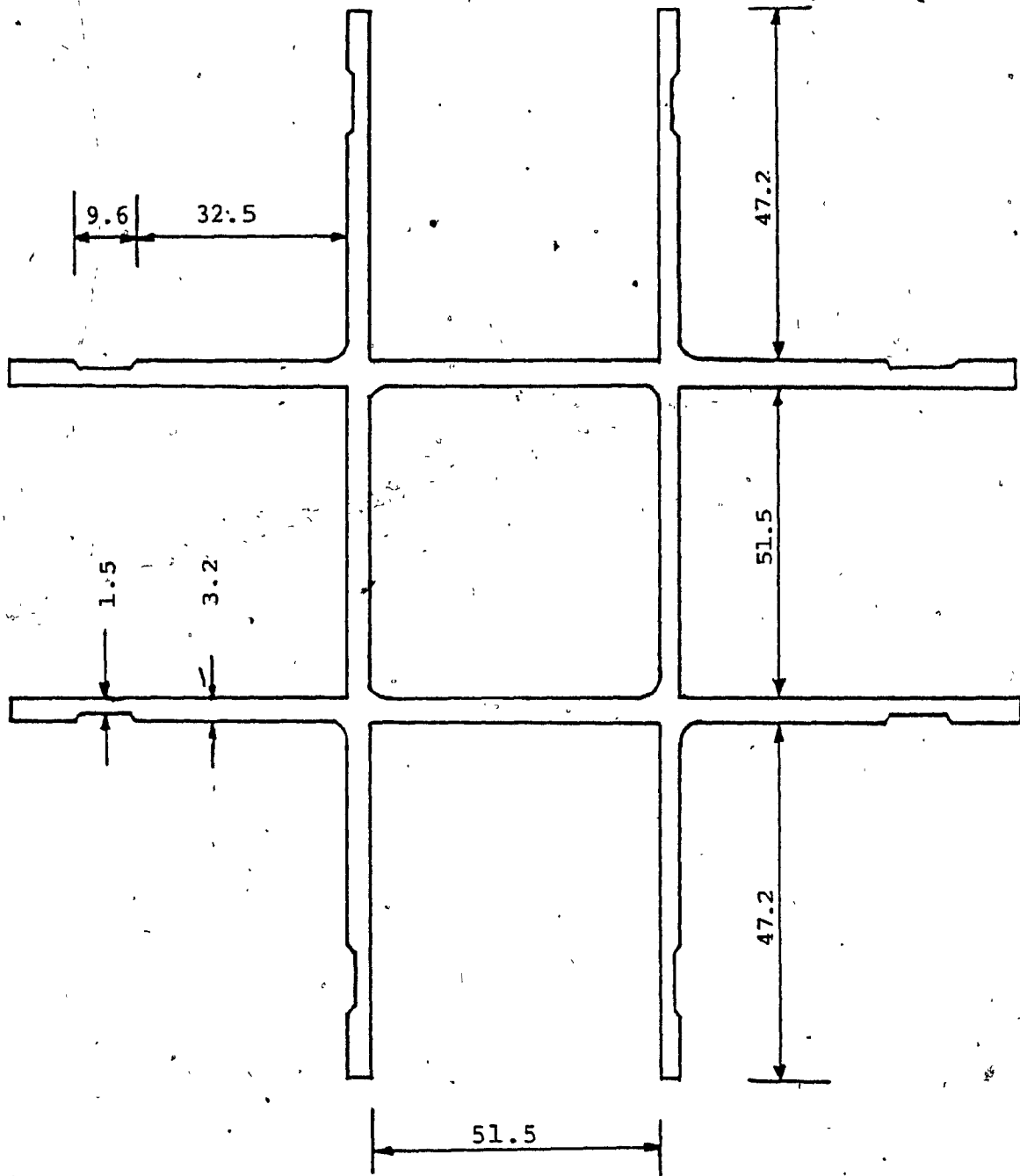
Fig. 2.1 - TYPE # EXTRUSION TO ACHIEVE
A FOUR WAY CONNECTION
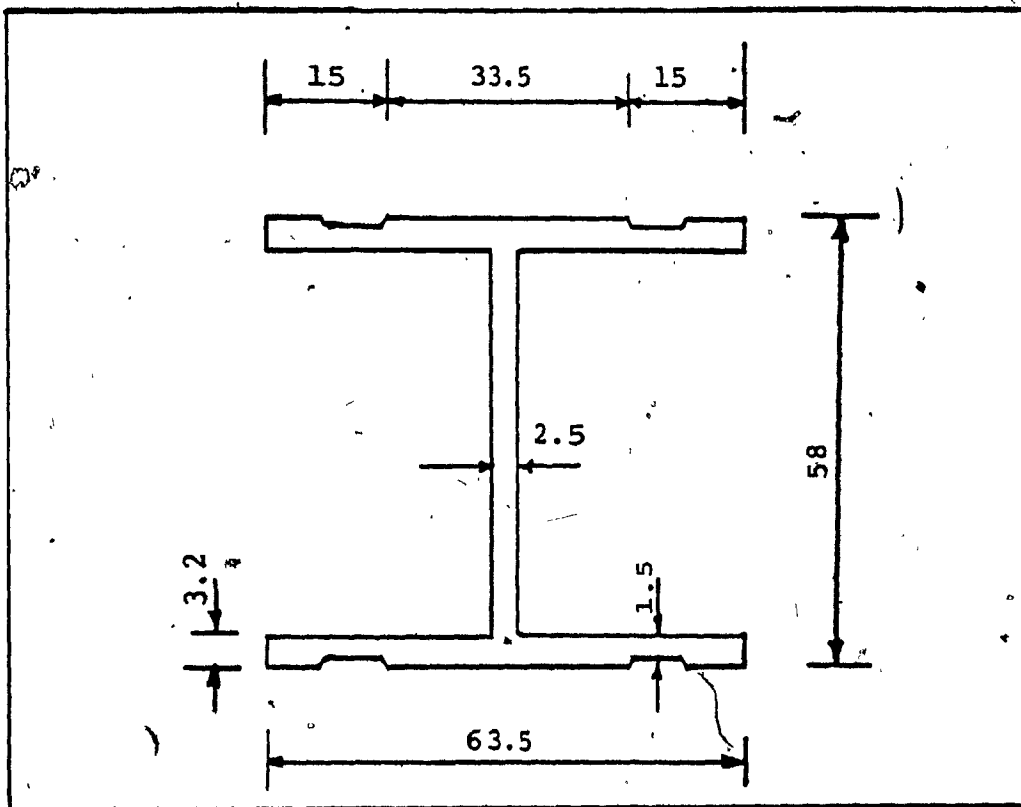
Fig. 2.2 - TYPE I EXTRUSION
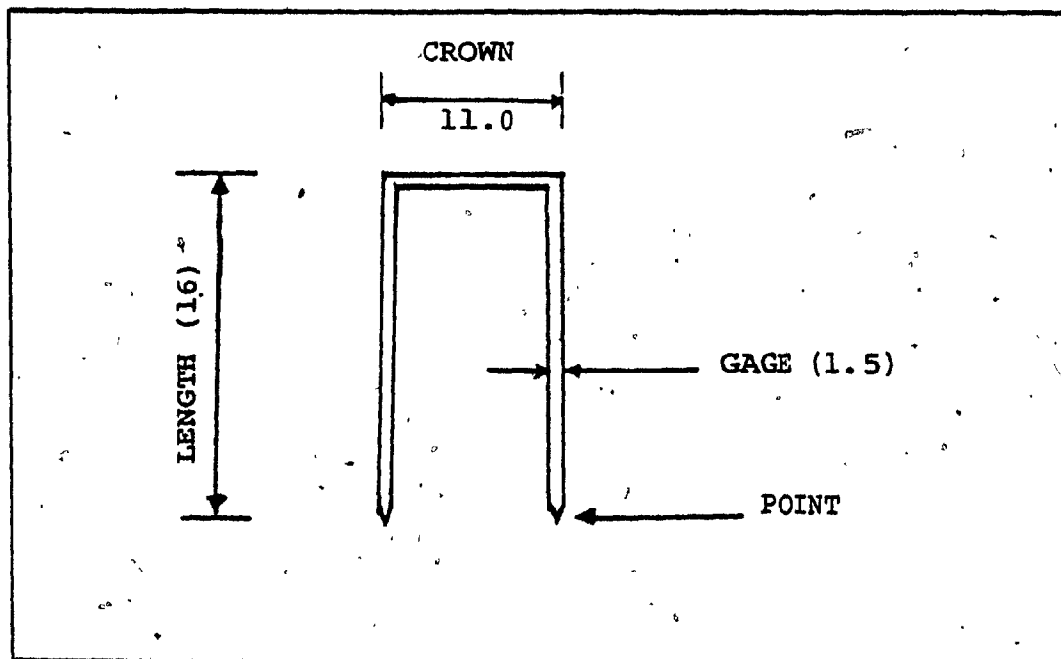
Fig. 2.3 - STAPLE SPECIFICATIONS
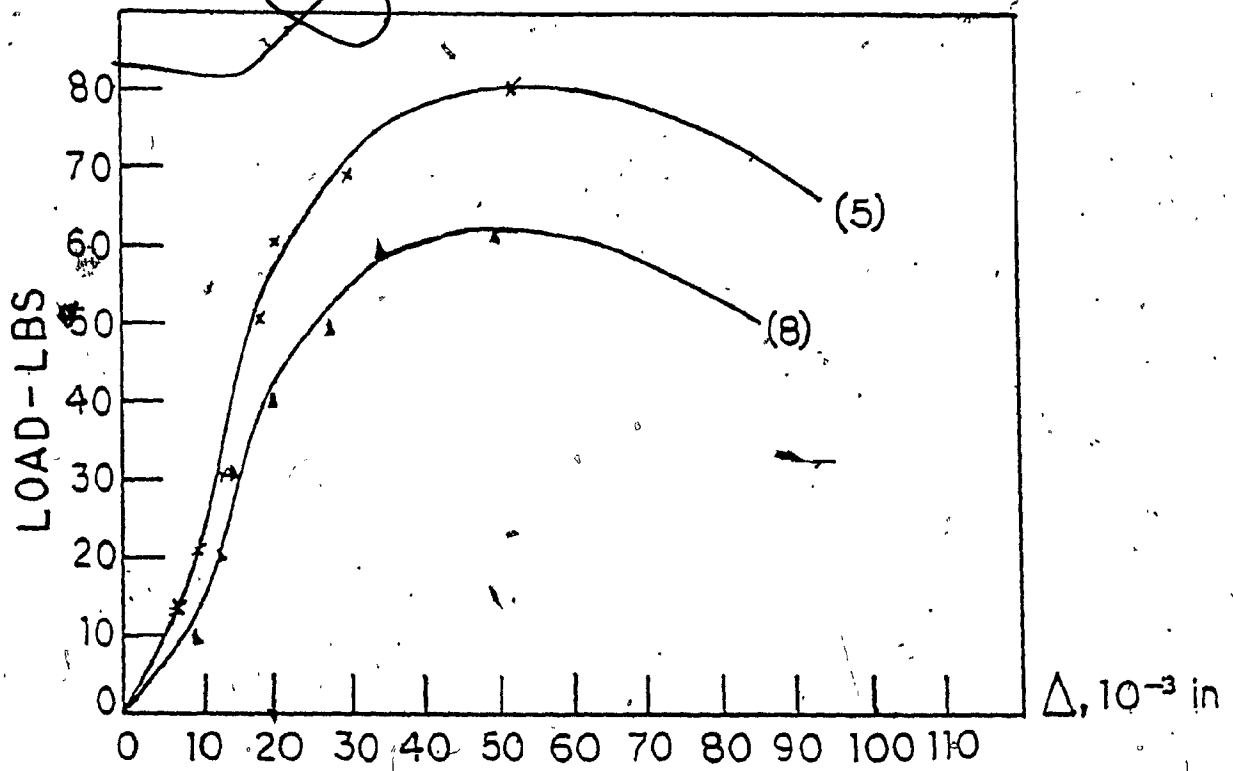
Fig. 2.4 – LOAD PER STAPLE LEG VERSUS THE DEFORMATION FOR DIFFERENT NUMBER OF STAPLES IN A ROW (17).

shim, 1/8 in. (3 mm) thick, was inserted in one side of the connection to eliminate the gap between panel and extrusion (Fig. 2.6).

The spacing of staples in each row is related to the calculated load per unit length at that location. The bottom rows had two staples per inch, the first level, one staple per inch; the second level, one staple per two inch; and the third level, one staple per three inch. All the vertical extrusions were connected to the panels with one staple per three inch.

Fig. 2.7 shows all the details and dimensions of the model.

2.2.1.4 Limited Slip Bolted Joints

The joints used were made of aluminium 5454-H32 plates with slotted holes, 3/4 in. (19 mm) long. The mechanical properties of the plate were measured and the results obtained are presented below:

Ultimate Stress F_u = 34 ksi   (230 MPa)

Yield Stress   Fy = 27 ksi   (190 MPa).

The plates were welded to "#" shaped extrusions, and bolted to "I" shaped extrusions through the slots (Fig. 2.8). Brake lining pads, 1/8 in. (3 mm) thick, were inserted between the plates and the "I" shaped extrusions to provide the required friction. They were made up of asbestos fibre reinforced with metallic particles. Since pads were not placed between plates and "#" extrusions, a gap was left. To eliminate this gap, washers with the same thickness of the pads
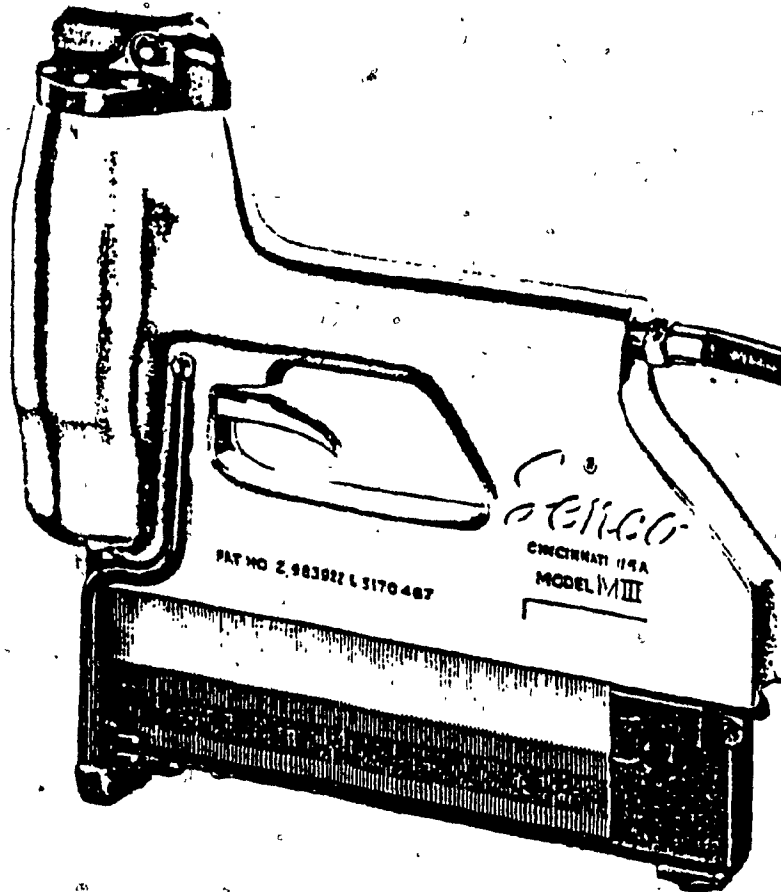
Fig. 2.5 – PENUMATIC STAPLER

PLYWOOD FILLER

| 10 | 43.5 | 10 |
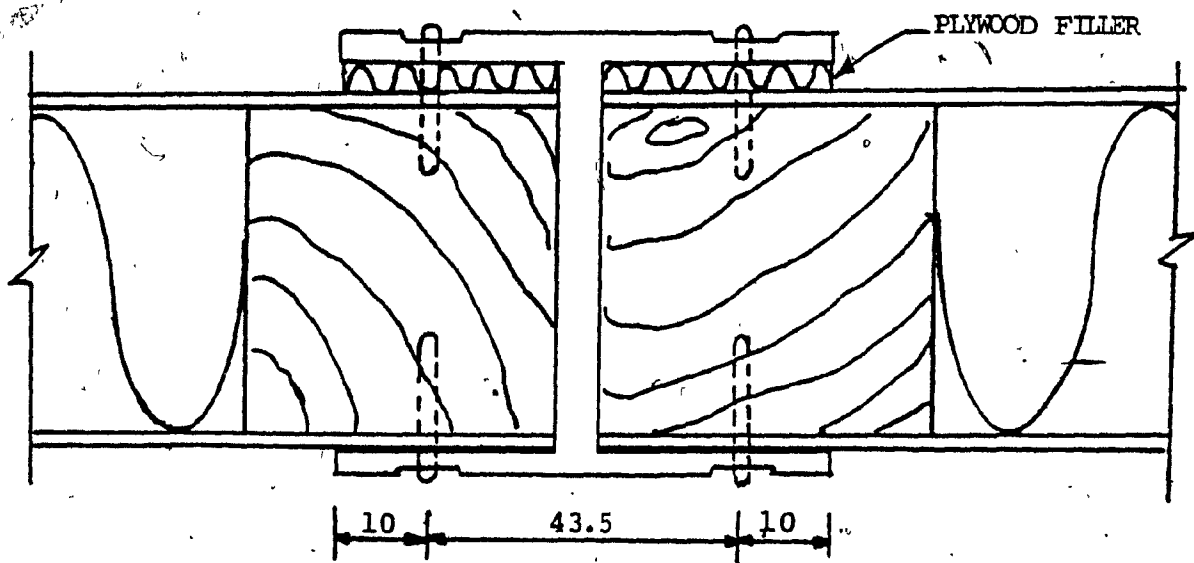
Fig. 2.6 – CROSS SECTION OF THE CONNECTION

610　　220　　610

3810

1220

50

Shaking Table

Fig. 2.7 - MODEL DIMENSIONS

Pads

Weld

Aluminum Plate

$\frac{1}{4}"$ Bolt

Brake- lining Pads

Double Nuts

Washers
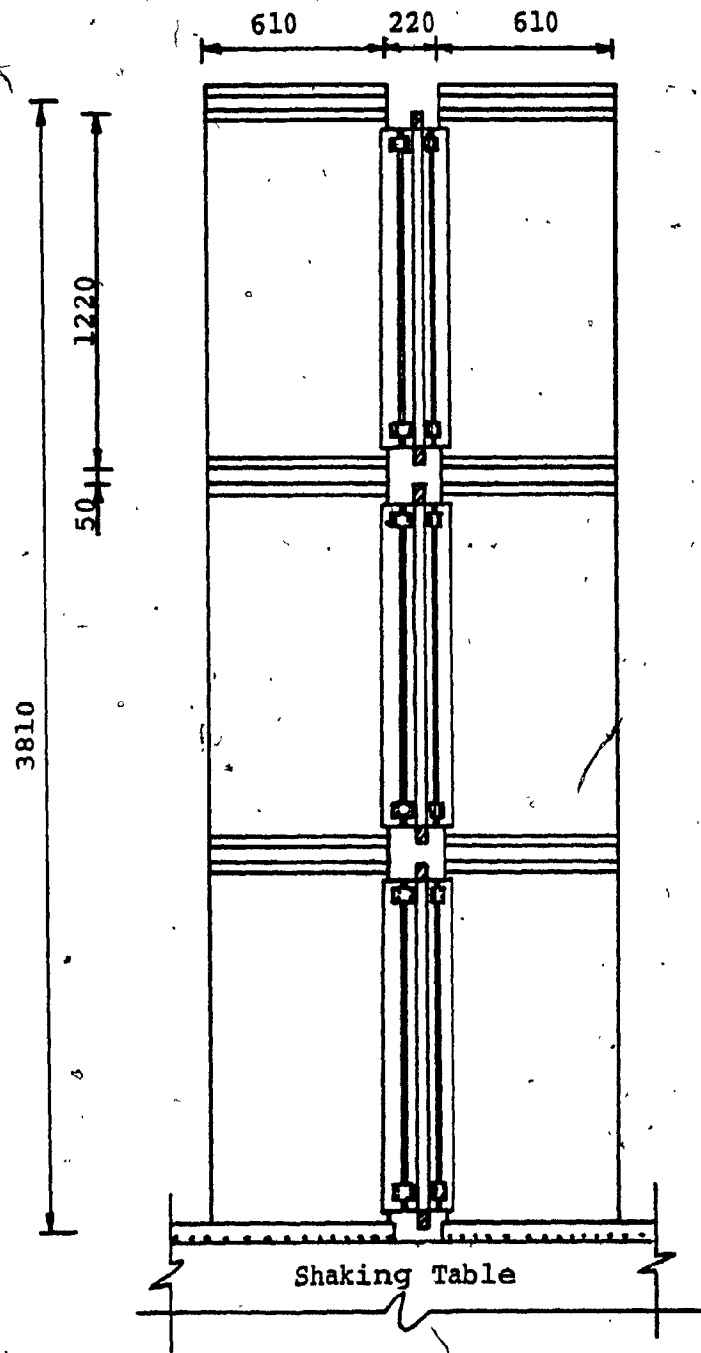
Aluminum Plate (used as washers)
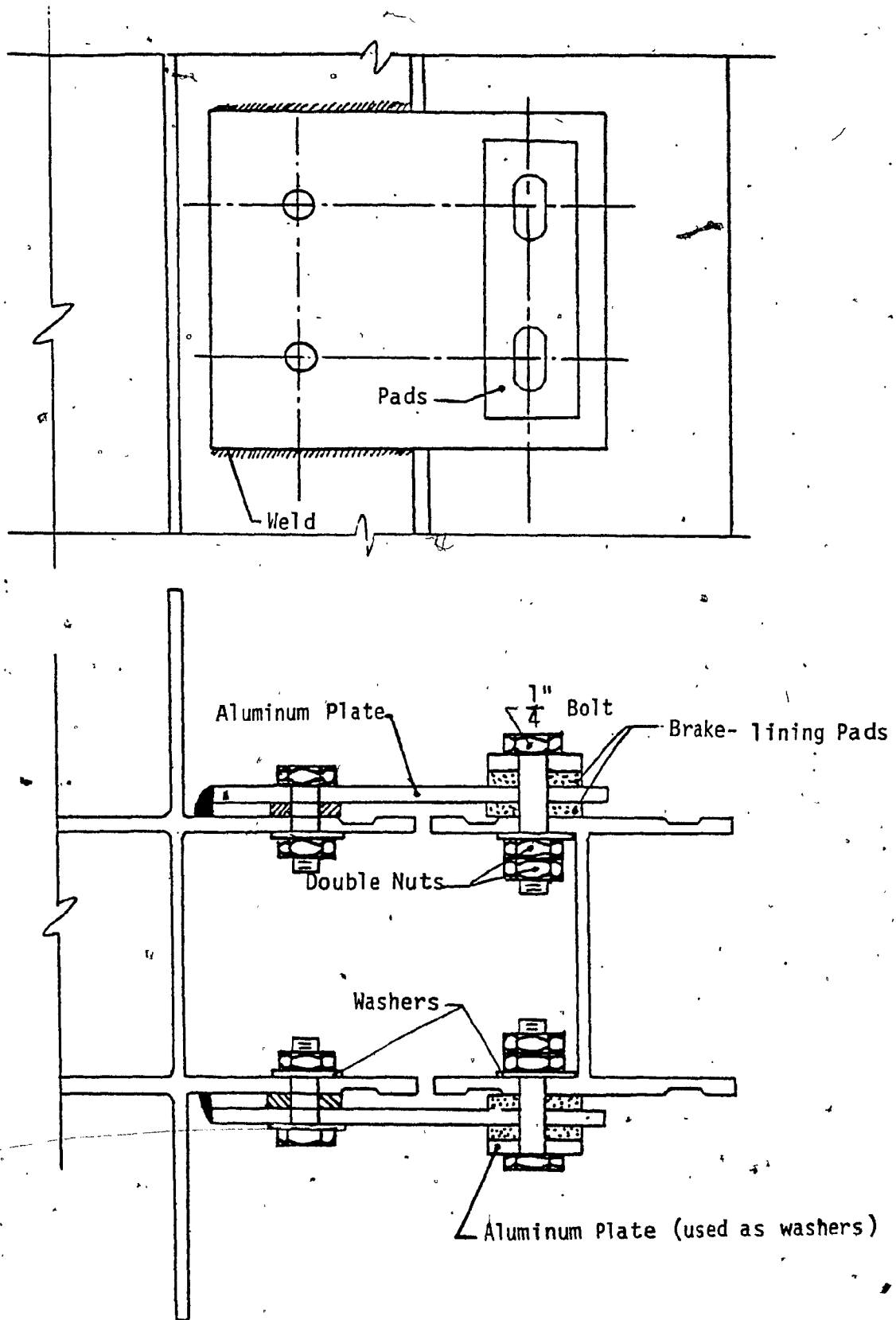
Fig. 2.8 Wall-to-Wall Joint of the Model (Specifications)

were used. Brake lining pads were also inserted between the external face of the plates and the bolt head to give the same coefficient of friction on both faces of the plate.

2.2.1.5 Cyclic Tests on the Joints

To find the hysteresis loop of the proposed joints a specimen (Fig. 2.9) was tested in an Instron universal testing machine, Model 1125. The specimen was attached to the load cell and the compressive or tensile loads were applied by the moving cross head. The Instrom strain gauge extensometer was connected directly to the servo chart drive system for a continuous load/displacement record.

The slip load of a friction bolted joint assembly depends on the clamping force caused by tightening of the nuts. Tests were done on different tightenings, using the Torque method and snug tight procedure. The best results were found by turning the nut in increments of 90 after they were tightened by hand. The corresponding hysteresis loop after 30 cycles is shown in Fig. 2.10.

It can be seen that the characteristic remains the same. The slip load per bolt had an average of 80 lbs. In the actual instalation in the model, the plates were welded to the "#" extrusions after the bolts were tightened to the "I" extrusions. This resulted in a residual slip load when the bolts were slackened, estimated at 10 pounds per bolts. Actual installation of the joints is shown in Fig. 2.11.

2.2.1.6 Connection Between the Model

and the Shaking Table

A steel frame with the same dimensions as the shaking
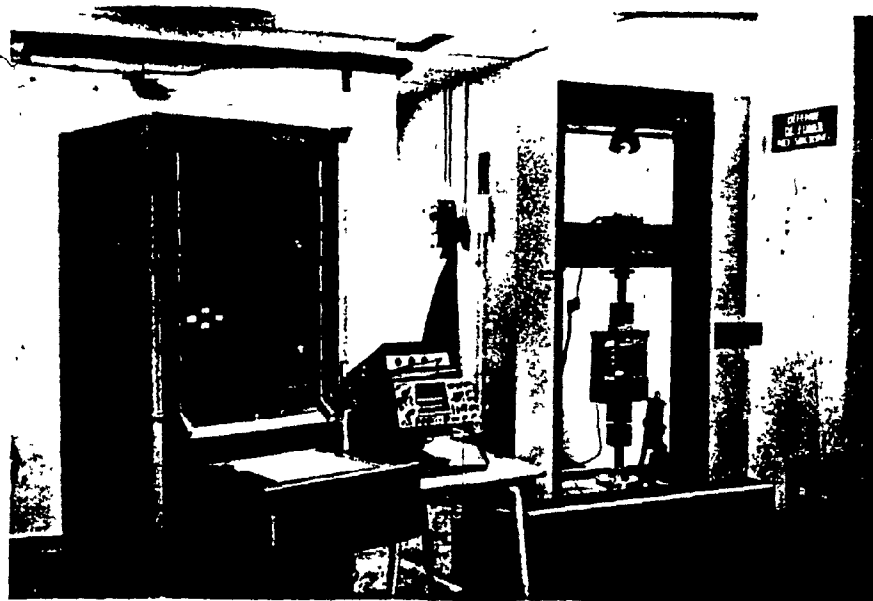
Fig. 2.9 - SPECIMAN IN THE INSTRON MACHINE
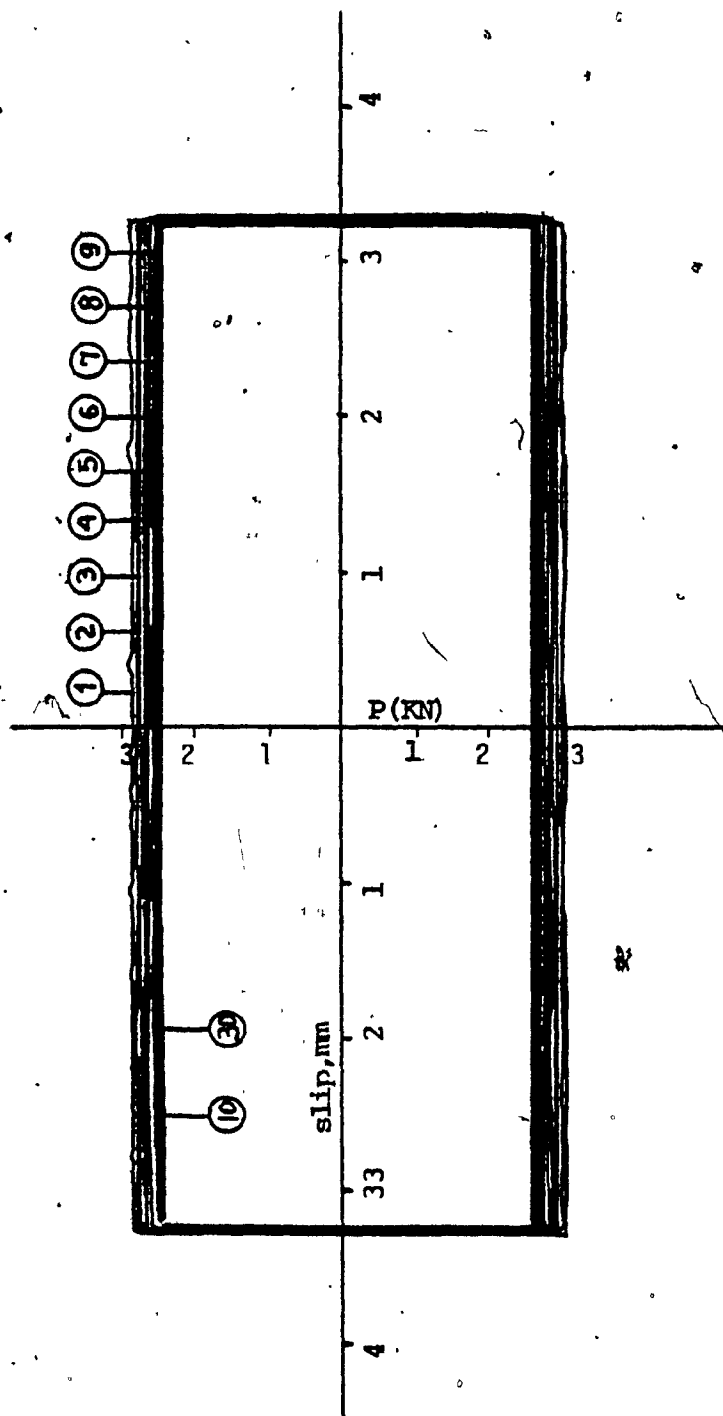
Fig. 2.10 - HYSTERESIS LOOP OF THE FRICTION JOINT USED IN THE MODEL
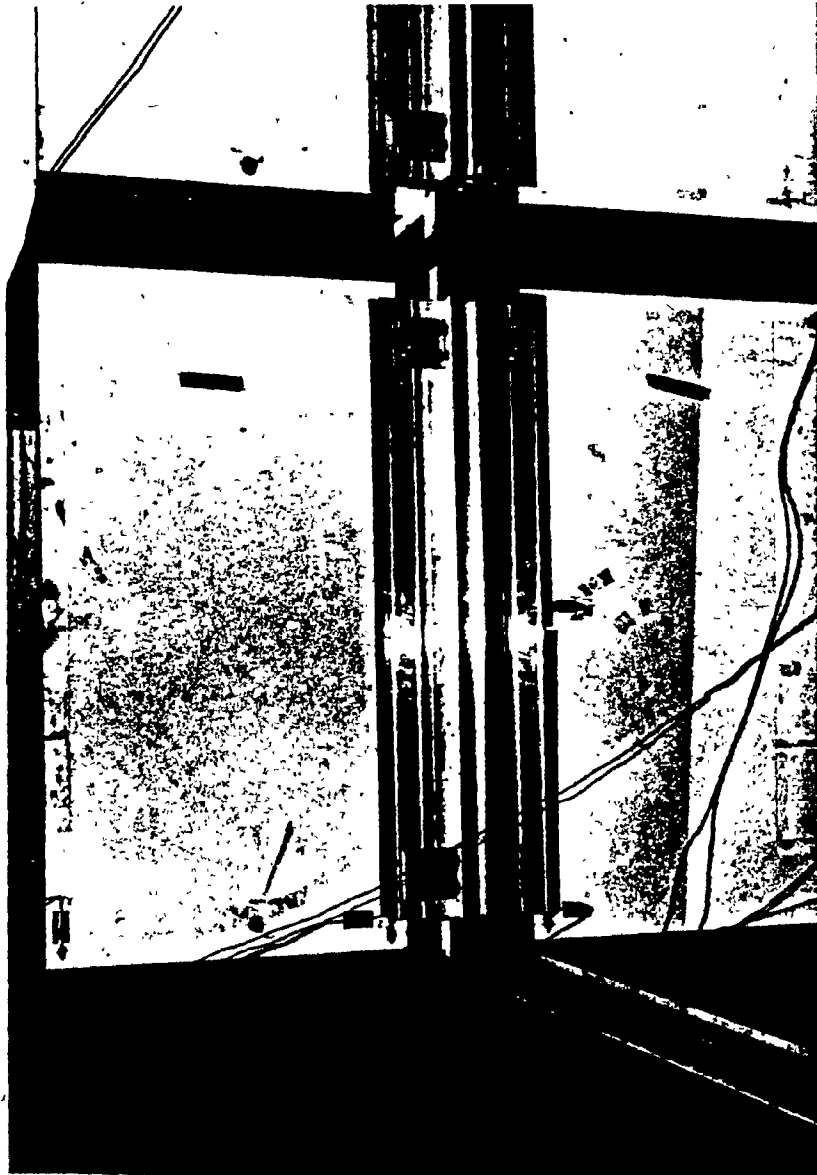
Fig. 2.11 - WALL TO WALL JOINTS OF THE MODEL

table 19 x 13 ft. (5800 x 3970 mm) was welded to the corners

of the shaking table. The connection of the model to the

frame is shown in Fig. 2.12.

2.2.1.7 Weight of the Model

The total weight of the model was 730 lbs. (330 kg).

The natural period was considered to be too low. To increase

the period of vibration and to help ensure that only the fun-

damental mode appeared, two steel channel shapes, each of

340 lbs. (155 kg), were connected to the top of the model

(Fig. 2.13).

2.2.2 Earthquate Facilities

Facilities to fulfill the requirements for simulating,

reading, and recording earthquake response can be divided

into five sub-systems.

1) Shaking Table

2) Hydraulic System

3) Electronic System

4) Additional Equipment for Static Tests

5) Data Collecting System.

2.2.2.1 Shaking Table

The shaking table available at Concordia University

has the dimensions of 13 ft. (4 m) in the direction of motion

and 19 ft. (5.8 m) in the direction perpendicular to the

motion. It is capable of 2/3 g (6.54 m/sec) acceleration,

25 in/sec (0.64 m/sec) maximum velocity and $^\pm$3 in. ($^\pm$76 mm)

maximum displacement (18).

The table is supported on a floor slab which is
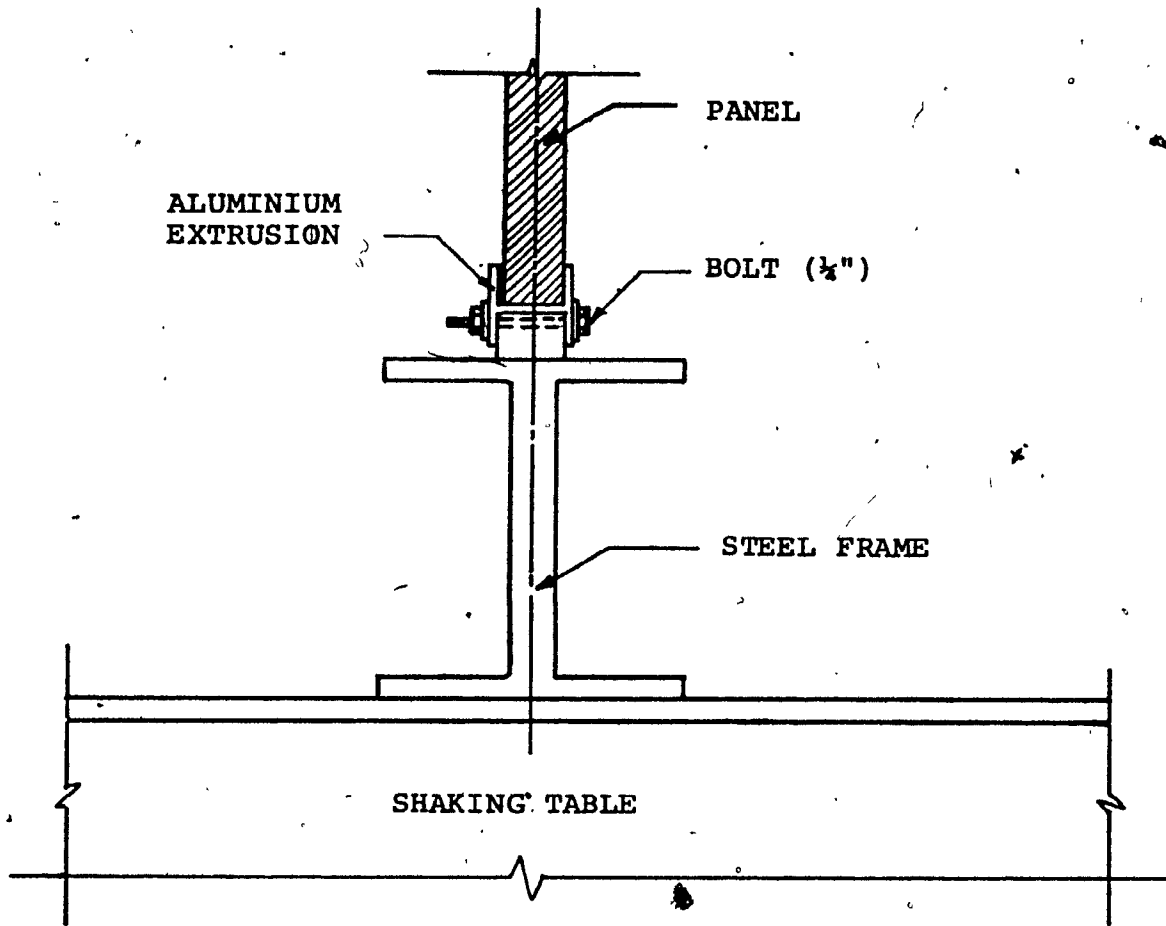
Fig. 2.12 – CONNECTION OF THE MODEL TO THE TABLE
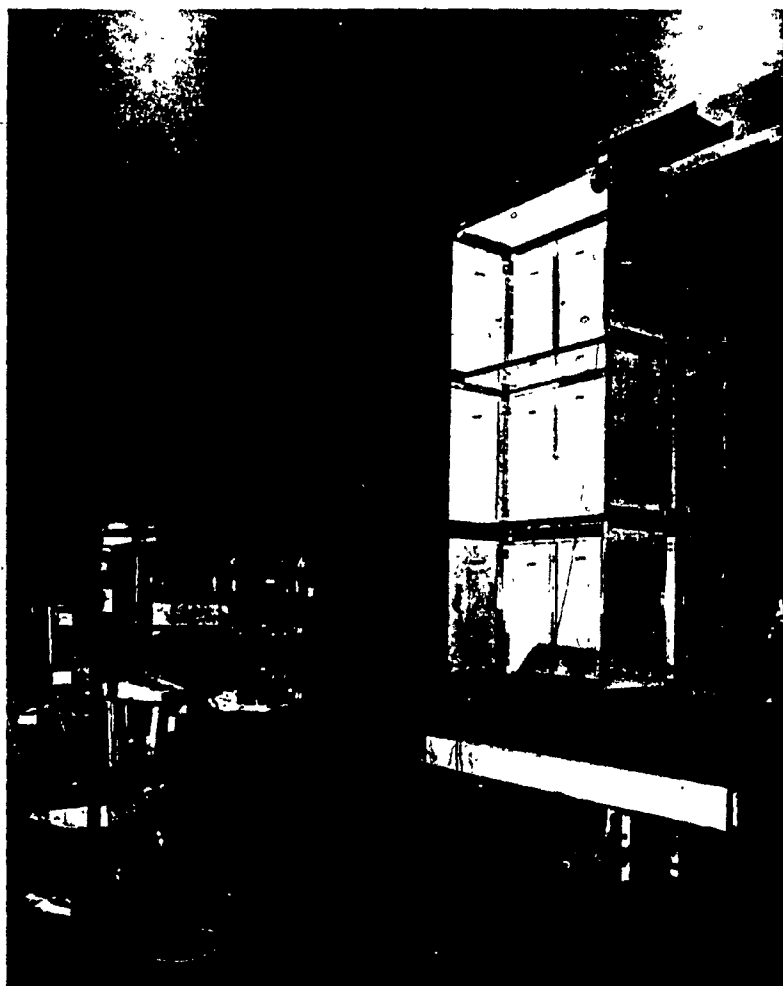
Fig. 2.13 - MODEL ON THE TABLE

anchored to rock (Fig. 2.14).

2.2.2.2 Hydraulic System

The hydraulic system available includes three main sections, a) control console, b) actuator, and c) hydraulic pumping unit.

a) The control console provides the means to control load, loading rate, displacement, frequency, amplitude, and wave form imposed by the actuator, Fig. 2.15.

b) Two actuators were available as a drive system, 20 kips (9. ton) and 50 kips (23 ton). Present test was done using the 20K actuator, which had an effective piston area of 9.5 in.^{2} (6100 mm^2) with a rod diameter of 2 in. (51 mm).

c) The hydraulic pumping system had a maximum capacity of .10 GPM, and a maximum pressure of 3000 psi.

2.2.2.3 Electronic System for

Operating the Shaking Table

The Gilmore Structural Loading System was used for operating the shaking table.

The control console consists of a servo amplifier, a signal conditioner, function generator with dynamic amplitude conditioner, rate programmer, and a digital frequency counter.

The actuator could be operated with position or load control.

2.2.2.4 Additional Equipment

for Static Tests

It is necessary to know the stiffness of the model for the analysis of the results. Since the shaking table is
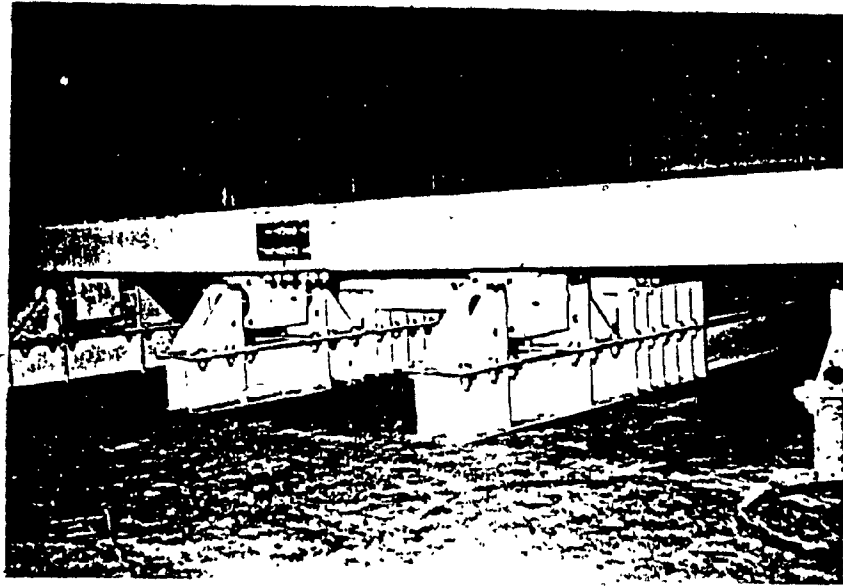
Fig. 2.14 - GENERAL VIEW OF TABLE
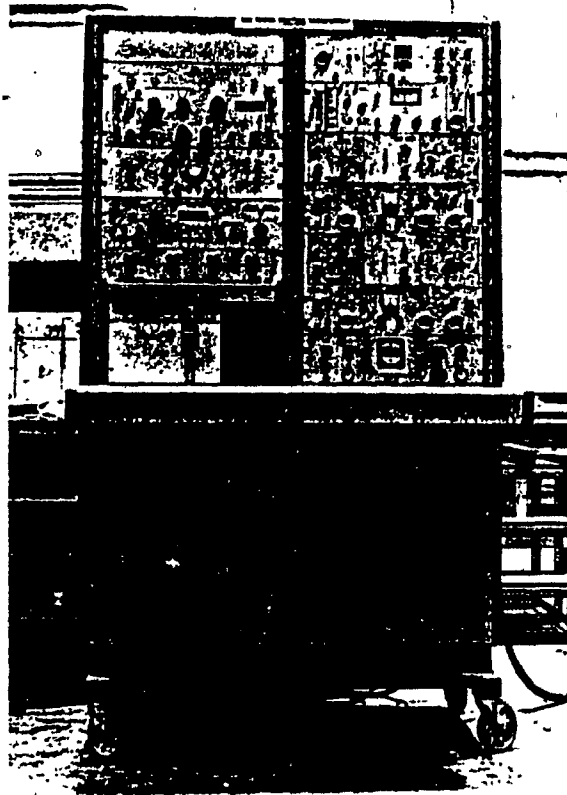
Fig. 2.15 - GILMORE CONTROL CONSOLE

situated approximately 3 ft. (.9m) from the concrete wall in the laboratory, the middle point of the top of the model was connected to the wall with a turn buckle, to transmit a horizontal point load. The load was applied by moving the shaking table away from the wall. By connecting a dynomometer to the turn buckle, the applied force was measured. The deflection on the top of the model was found with a dial gage, and the deflection versus force curve plotted, from which the stiffness was calculated.

2.2.2.5 Data Collection Systems

Different data collection systems were used, depending on the kind of measurement required. These measurements and their corresponding systems are described briefly below (Fig. 2.16).

2.2.2.5.1 Strain Measurements

Eight strain gages were connected to the four sides of the coupled walls, where the stress was a maximum. The strain gages were (EA-13-100 BR-120, made by #M#M#). Since a dynamic reading was required, they were connected to an electronic voltmeter made by B & K, type 2409/2416. This instrument is able to measure the peak, average absolute, and true RMS values of AC voltages in the frequency range of 2-200,000 HZ.

2.2.2.5.2 Oscilloscope

A Tektronix 7633/R7633 (Fig. 2.17) storage oscilloscope was used to make the wave forms visible on a fluorescent screen. It is a solid-state instrument, especially designed for fast writing rate storage applications. The 7633 operates
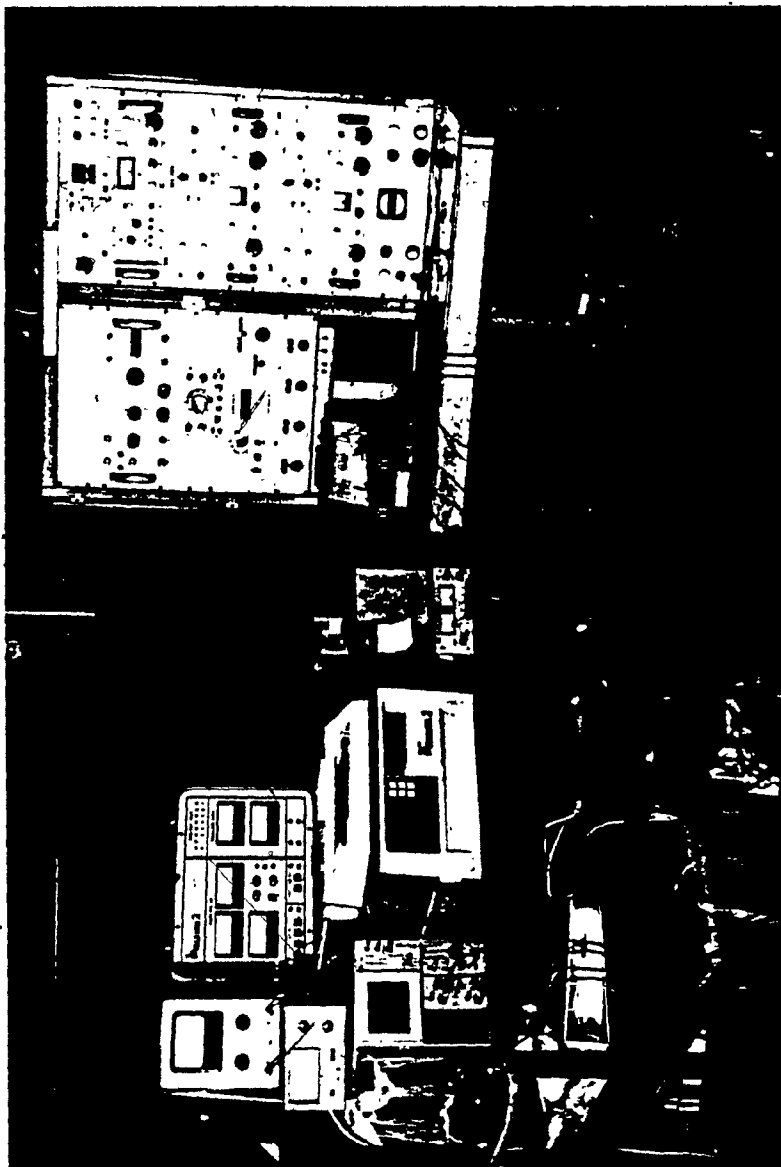
Fig. 2.16 - DATA COLLECTION SYSTEMS
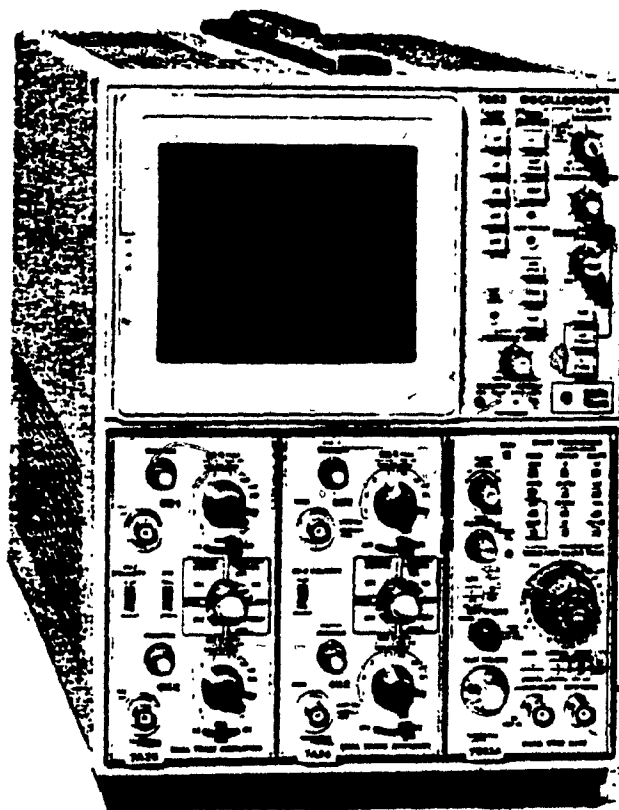
Fig. 2.17 – OSCILLOSCOPE

in three display modes, NONSTORE, STORE and SAVE.

The trigger source switch allows selection of the internal trigger signal for the time base unit.

The two inputs used in the tests were the displacement on top of the model, and the displacement of the shaking table. The phase between two motions and their characteristics were obtained from the screen and a phasemeter.

2.2.2.5.3 Displacement Measurements (Amplitude)

The amplitudes of the shaking table and of the top of of the model were measured by a displacement transducer composed of a strain gage (CEA-06-125VW-350, made by =M=M=) connected to a thin steel cantilever, which followed the motion. For frequencies more than 2 HZ the peak value was read by the B & K, electronic voltmeter type 2409/2416, which was also used for the strain measurements in the panels. For frequencies less than 2 HZ the measurements were done by a digital Multimeter type 3466A made by Hewlett Packard (hp).

The slipping of the uppermost plate was also measured by this procedure.

The phase angle between the top of the model and the shaking table was read by the B&K, phasemeter type 2971, from it the exact relative displacement was calculated.

CHAPTER III

TEST PROCEDURE AND RESULTS

3.1 Optimization

During earthquakes the buildings oscillate with an amplitude proportional to the input energy. To optimize the created response due to seismic forces, the input energy must be minimized, or the energy dissipated maximized. For a given motion, the input energy is mainly dependent on the natural frequency of the structure. The natural frequency of a structure depends on its stiffness and mass. The overall stiffness of the coupled walls is related to the strength and stiffness of the vertical joints, which in the case of LSB joints means the slip load. Thus the slip load is the major factor affecting the frequency of vibration, which in turn influences the input energy and the energy dissipation. A coupled wall with strong joints between the two walls, creating a solid wall has a higher stiffness than the isolated walls with zero slip load. The natural period of vibration of the model are shown in Table 3.1.

Type of Wall	Isolated Walls	Optimum Coupled Walls with LSB Joints	Monolithic Wall
Natural Period (Seconds)	.30	.26	.15

Table 3.1 The Natural Periods of Vibrations

The energy dissipation in LSB joints is a function of slip load and distance travelled, which will depend on the response of the building to the ground motion. Thus the

optimum solution is related to the intensity of the ground motion.

For this study comparisons were based on resonant ground motion with a constant maximum acceleration, representing the worst condition for a given intensity of ground motion.

3.2 Test Procedure

To find the optimum value tests were performed for different slip loads. The slip loads were obtained in the following manner: a) the bolts were tightened in the manner described in section 2.2.1.5, giving 80 lb/bolt (360N) for the slip load; b) the bolts were loosened, giving an estimated 10 lb/bolt (45N), as was described in section 2.2.1.5; c) the pads between the joints were removed, so the slip load was zero.

The bed was driven by a sinusoidal force with a maximum value equal to 800 lbs (3600N), which created a maximum acceleration of 30.5 in/s^2 (77.5 cm/s^2) kept constant for all the tests. For each test the frequency of the sinusoidal motion of the shaking table was adjusted to generate resonance of the model. The frequency and the amplitude of the shaking table and the top of the model were measured. Different tests and their corresponding results are described below:

3.2.1 Test Program

Test I

All the bolts in LSB joints were tightened, creating a slip load equal to 1280 lbs/storey (5700N), 80 lb/bolt

(360N) in each wall. The static test for measuring the stiffness of the model was carried out.

By moving the table away from the wall while the top of the model was connected to the wall, the force and the displacement of the top of the model were measured.

Dynamic tests were done using the procedures presented earlier. Frequency, amplitudes of the bed and the top of the model and maximum stress level were measured at resonance. Since there was no slip in the LSB joints this case can be considered as monolithic walls.

Test II

Each panel was connected by two LSB joints at the top and bottom on both sides. The bolts at the bottom joints were loosened in each panel creating a slip load of 720 lbs/storey (3200N), (80 lb/bolt (360N) for tightened bolts and 10 lb/bolt (45N) for loosened ones) in each wall.

Test III

The two bolts at the bottom joints were loosened as well as one bolt at the top joints. The slip load was 400 lb/storey (2000N),(80 lb/bolt (360N) for tightened bolts and 10 lb/bolt (45N) for loosened ones).

Test IV

This test was done while all the bolts in each panel were loosened, creating a slip load equal to 160 lbs/storey (720N), (10 lb/bolt (45N) in each wall).

Test V (Isolated Walls)

All the pads between the joints were removed creating isolated walls.

3.3   Test Results and Discussion

For each of the values of the slip load Table 3.2 gives the natural frequency, the amplitudes of the bed and the top of the model, and the maximum stress level.

These values are plotted in Figs. 3.2 to 3.5. Fig. 3.1 shows the deflection versus force curve from static test.

From the study of Table 3.2 and Fig. 3.2, it can be seen that a slip load of 160 lb (720N) provides the minimum deflection of the model. Fig. 3.3 shows the effect of different slip loads on top deflections of the model with respect to the monolithic wall.

The relationship between slip load and the maximum stress is similar and is represented in Fig. 3.4 which shows a slip load of 160 lb (720N) gives the optimum result. The effect of different slip loads with respect to monolithic wall is shown in Fig. 3.5.

P_S (N)	T (Sec)	A_{top} (mm)	σ (Psi)	A_{bed} (mm)
0	0.30	5.08	20,700	1.78
720	0.26	0.76	4,700	1.27
2000	0.17	1.78	13,100	0.58
3200	0.16	2.54	18,600	0.51
5700	0.15	2.79	20,000	0.43

TABLE 3.2 — EXPERIMENTAL RESULTS
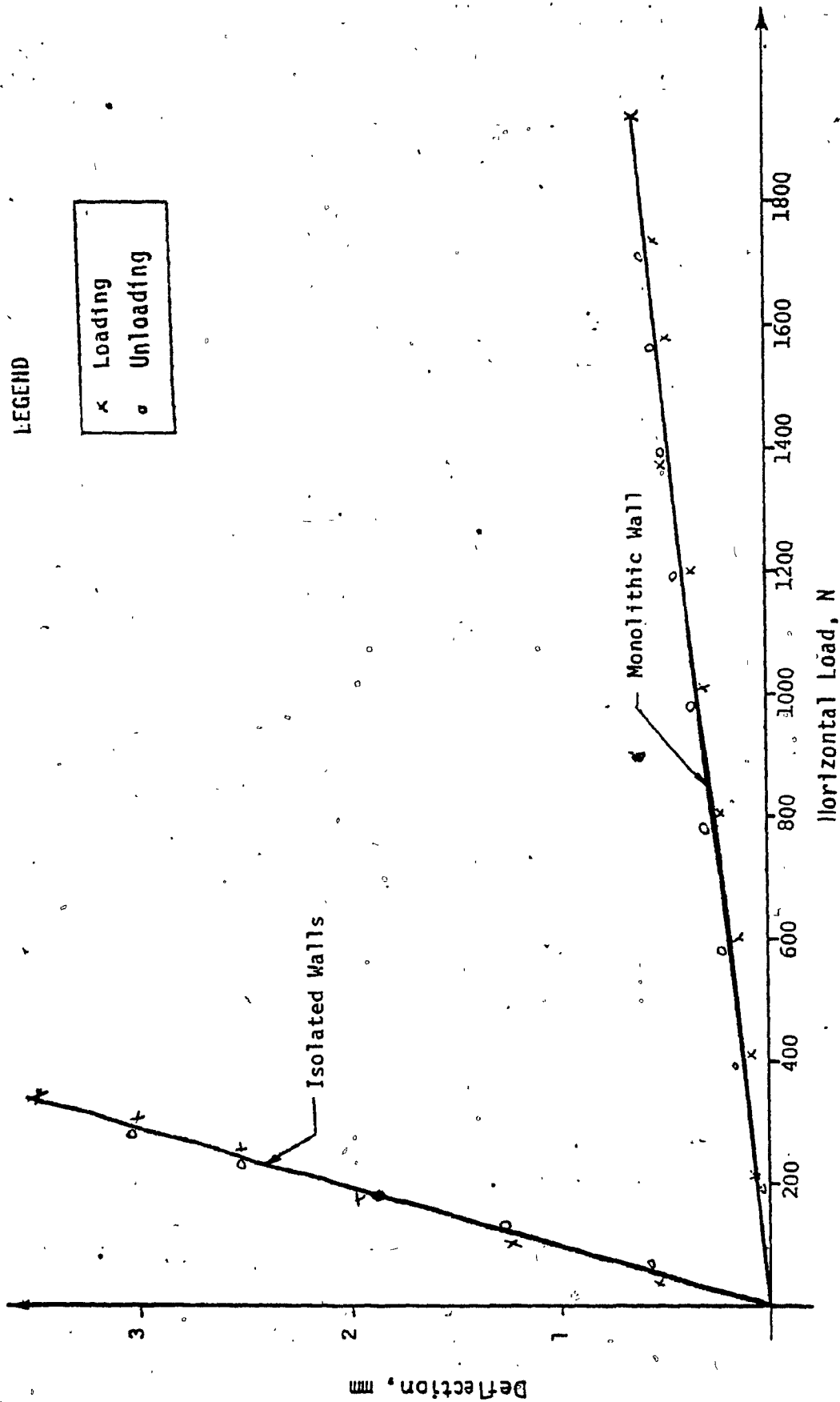
Fig. 3.1. P – Δ Relationship of the model
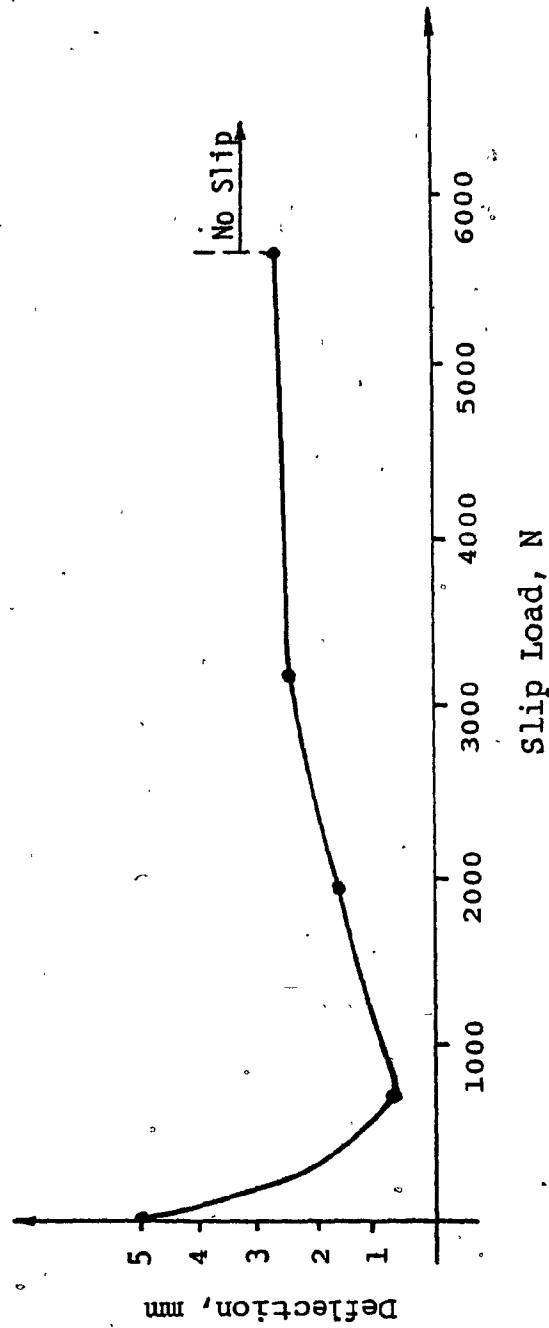
Fig. 3.2 - EFFECT OF SLIP LOAD ON DEFLECTIONS
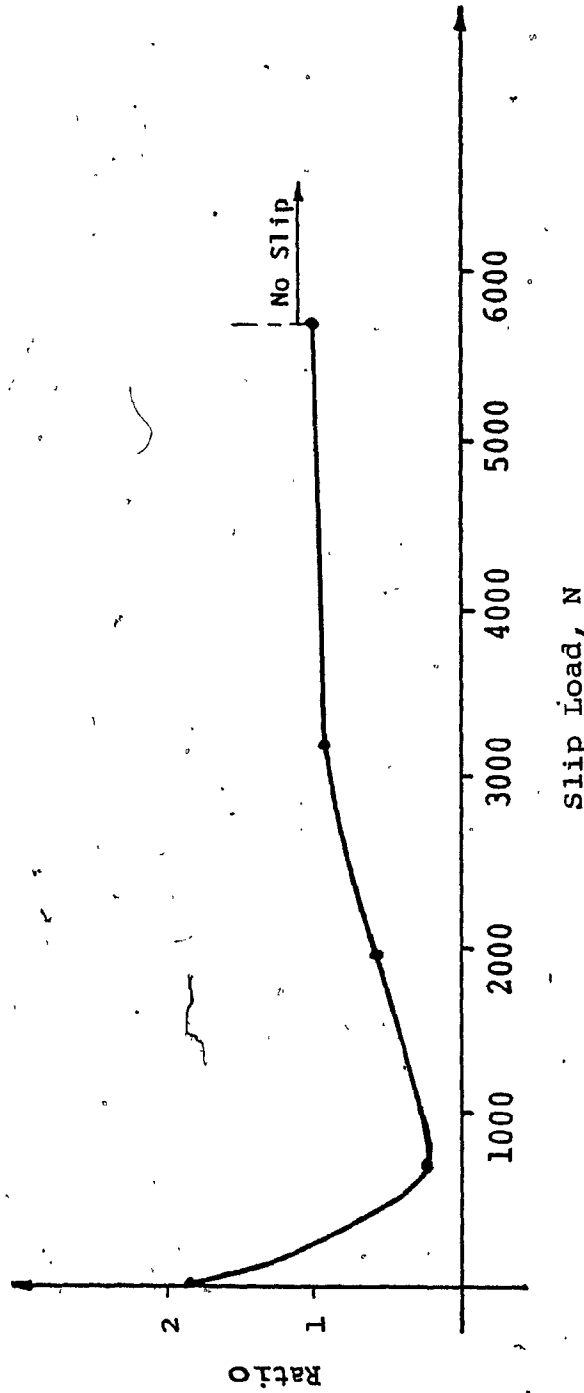
Fig. 3.3 - RATIO OF DEFLECTION OF JOINTED WALL TO MONOLITHIC WALL
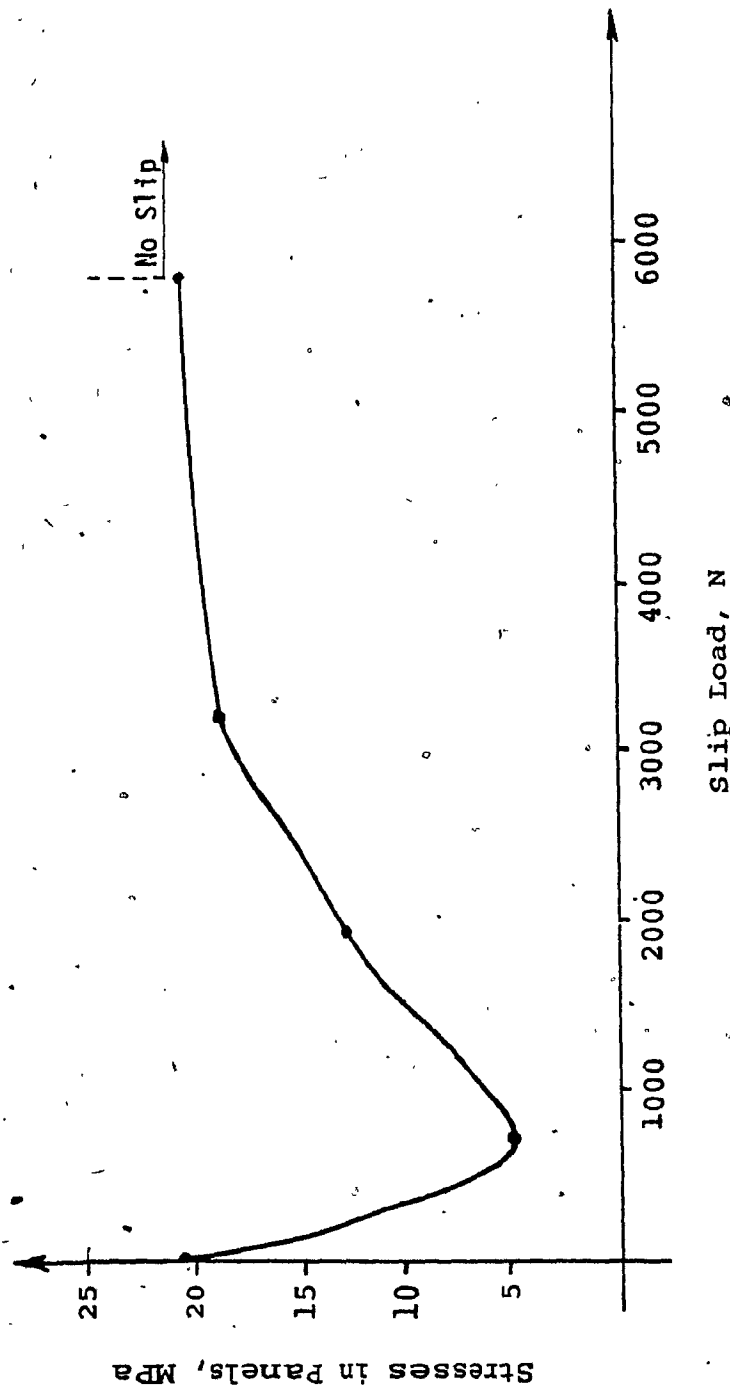
Fig. 3.4 - EFFECT OF SLIP LOAD ON STRESSES
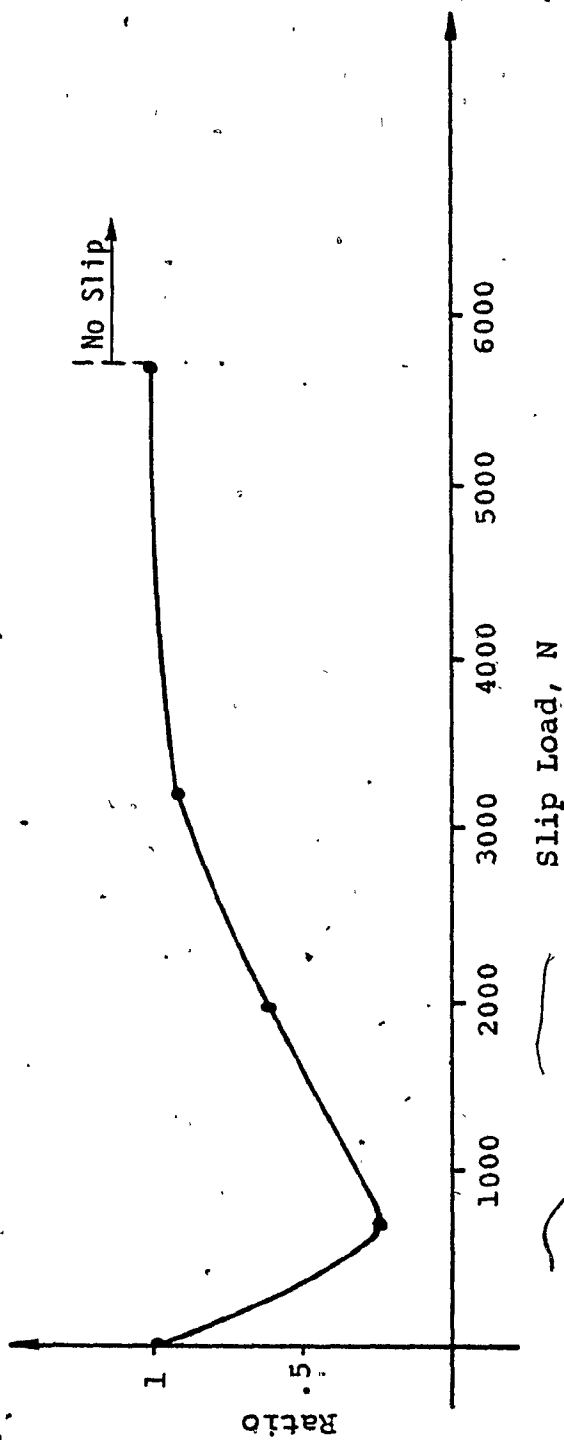
Fig. 3.5 - RATIO OF STRESS IN JOINTED WALL TO MONOLITHIC WALL

CHAPTER IV

ANALYSIS

4.1 Methods of Analysis

An important aspect in the analysis of a structure is the choice of a proper model. The proposed model must consider the paramount importance of each working component. The main methods of modeling the shear walls are the finite element, the frame analogy, the beam model, and the continuous medium.

The finite element method is able to handle static and dynamic behavior of shear walls in linear or nonlinear stages and the method has been widely used (19, 20, 21). The accuracy of the method relies on the fineness of the mesh which affects the cost of analysis. The output is in the form of stresses.

Frame analogy method is a convenient method since it can be used easily by a standard plane frame computer and the output is in the form of moments, forces and shears. In this method the walls and beams are represented by their centroidal axis, with corresponding stiffness. The method can handle dynamic behavior of structures in linear or nonlinear phases and has been widely used in structures (22, 23, 24).

Beam model is a simple and fast technique for hand calculations but it cannot handle the dynamic behavior in nonlinear stage.

Continuous medium method, has been extensively used (25, 26, 26, 28) in the analysis of shear walls. The solutions in the form of curves and tables for different kinds of loading are available (25, 27) for fast calculations. This

method is simple and effective but is not usually used for dynamic analysis.

4.2 Proposed Model

Each of the aforementioned techniques has its own quality and limitations and the selected method depends on the desired accuracy. Since the desired accuracy influences the computer cost, this is another factor that must be considered.

To compare the results obtained from the test done with the theoretical values, an accurate analysis is required. This needs a nonlinear dynamic behavior, so there are two techniques available, finite element and frame analogy method. A study was done (29) to show the accuracy of frame analogy method compared with the finite element method using a fine mesh. The results showed a good coincidence with frame analogy method.

In another study (30) results obtained from a test done on a ten-storey structural model excited by means of a shaking table showed that the best results of different methods of analysis were found by frame analogy method. Since the object of the present research is to show the participation of LSB joints, in improving the seismic resistance of structures, it is important that the proposed method be able to consider it. Of most importance is the availability of an appropriate computer program. One using the frame analogy method was available.

4.3 Analysis Procedure

Most likely the step-by-step integration procedure is the most powerful method available for nonlinear analysis. In this method, the response is found for short equal time increments Δt and the calculated response at the end of each interval is considered as the initial condition for the next interval. The process is continued step by step from the beginning of loading to any desired time.

At any instant of time 't' the equation of dynamic equilibrium can be written as:

$$[M] \{d\ddot{r}\} + [C_T] \{d\dot{r}\} + [K_T] \{dr\} = \{dP\} \qquad (4.1)$$

where $\{d\ddot{r}\}$, $\{d\dot{r}\}$ and $\{dr\}$ are the increments of acceleration, velocity and displacement, respectively, at the nodes; $\{dP\}$ is the increment in applied loading, $[M]$ is the mass matrix and $[C_T]$ and $[K_T]$ are the tangent values of the damping and stiffness matrices for the structure in its current state. For a finite time step, Δt, the equation (4.1) can be written as:

$$[M] \{\Delta\ddot{r}\} + [C_T] \{\Delta\dot{r}\} + [K_T] \{\Delta r\} = \{\Delta P\} \qquad (4.2)$$

in which $\{\Delta\ddot{r}\}$, $\{\Delta\dot{r}\}$, $\{\Delta r\}$ and $\{\Delta P\}$ are the finite increments of acceleration, velocity, displacement and load respectively. The tangent damping and stiffness matrices are defined at the beginning of time intervals. $\{\Delta P\}$ is the increment of load and is equal to:

$$\{\Delta P\} = - [M] \{1\} \Delta\ddot{x} \qquad (4.3)$$

in which [M] is the mass matrix and $\Delta\ddot{x}$ is the increment of the ground acceleration. The method of Drain-2D is based on a linear variation of acceleration. The accuracy depends on the ratio of the time step and the period (31), and greater accuracy can be expected as the integration time step is reduced. In general, the time interval, Δt, should not be longer than one-tenth of the fundamental period (32).

4.4 Computer Program

The computer program used for the theoretical analysis was "Drain-2D" (33) developed at the University of California, Berkley. For step-by-step time integration of the dynamic equilibrium equations, the program consists of a series of subroutines. The direct stiffness method is used for the analysis with nodal displacements as unknown. Subroutines for arbitrary oriented truss elements, arbitrary oriented beam column elements, infill shear panel elements and semi-rigid connection elements are available. To represent the LSB joint behavior the subroutine for truss elements was mod-ified by A.S. Pall (1).

The time history of ground acceleration is considered for earthquake excitation. Lumped mass at nodes is viewed as structural mass.

4.5 Modeling Assumptions

The following assumptions are made for the proposed model on the shaking table, in order to compare the experi-mental results with the theoretical analysis.

The two shear walls connected with floor panels are considered as a plane structure.

The vertical panel walls are considered as continuous elastic cantilevers. It is assumed that the horizontal connections ("#" extrusions) prevent any shear slip or rocking type motion.

The floor diaphragms are considered to be infinitely rigid in their own plane.

The vertical panels are assumed to remain in their elastic range and their corresponding stiffness was found by the static test.

Only LSB joints are considered to have nonlinear behavior.

The mass dependent type viscous damping is assumed corresponding to 3% of critical damping for elastic panel members.

A single equivalent lumped mass was placed at the top of the structure in order to ensure the response in the fundamental mode.

In the proposed modeling procedure the coupled walls are idealized as an equivalent wide column frame, in which the panel walls are represented by their centroidal axis with their corresponding properties. The two columns are coupled together at each storey by rigid arms connected by LSB joints.

LSB joints are modeled as truss elements (1) yielding in the vertical direction in tension or compression to give elastic-plastic behavior.

Since in the experiment the shaking table gave a sinu-
soidal motion, so the input acceleration in the computer
analysis was in sinusoidal form. In each test the period of
vibration for the input acceleration was the resonant period.

The peak acceleration was the same as in the tests
(30.5 in/sec^2).

4.6 Results

The study was conducted for different slip loads with
the corresponding natural frequency. The results of these
studies are shown in Table 4.1 in respect to the maximum
stresses and deflection at the top.

Fig. 4.1 shows the effect of different slip loads on
top deflection. It can be seen that the slip load of 110 lb
(490N) gives the optimum solution. Fig. 4.2 also shows the
effect of different slip load on top deflections of the model
with respect to monolithic wall.

The effect of different slip load on stress level is
shown in Fig. 4.3 which shows that LSB-110 lb (490N) is the
optimum. The ratio of the stress level of jointed wall to
monolithic wall is shown in Fig. 4.4.

Figs. 4.6 and 4.7 show the maximum stresses and top
deflections obtained from the test and the computer analysis.

It can be seen that there is a close agreement between
the both results.

4.7 Approximate Analysis

The main reason for the current study is to demonstrate

P_S (N)	T (SEC)	A_{top} (mm)	σ (Ps1)
0	0.26	4.19	19,720
400	0.23	1.98	10,590
490	0.23	0.48	4,165
580	0.23	0.51	4,558
1220	0.14	1.07	10,390
1780	0.13	1.42	13,742
3100	0.12	2.51	19,582
5800	0.12	2.51	19,582

TABLE 4.1 - COMPUTER RESULTS

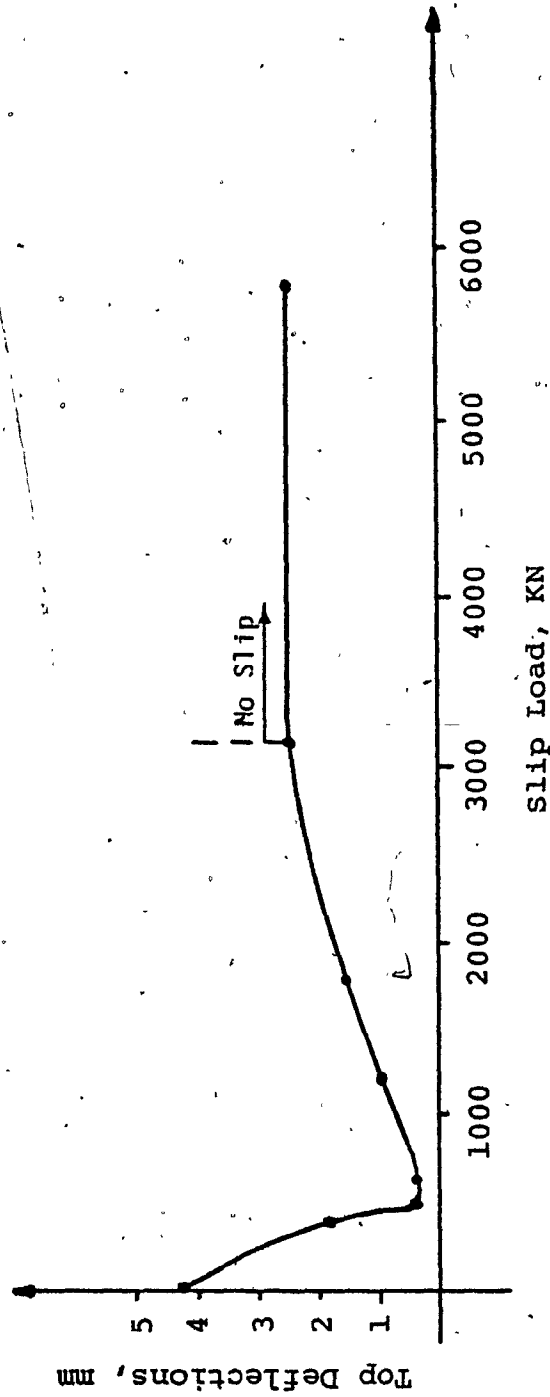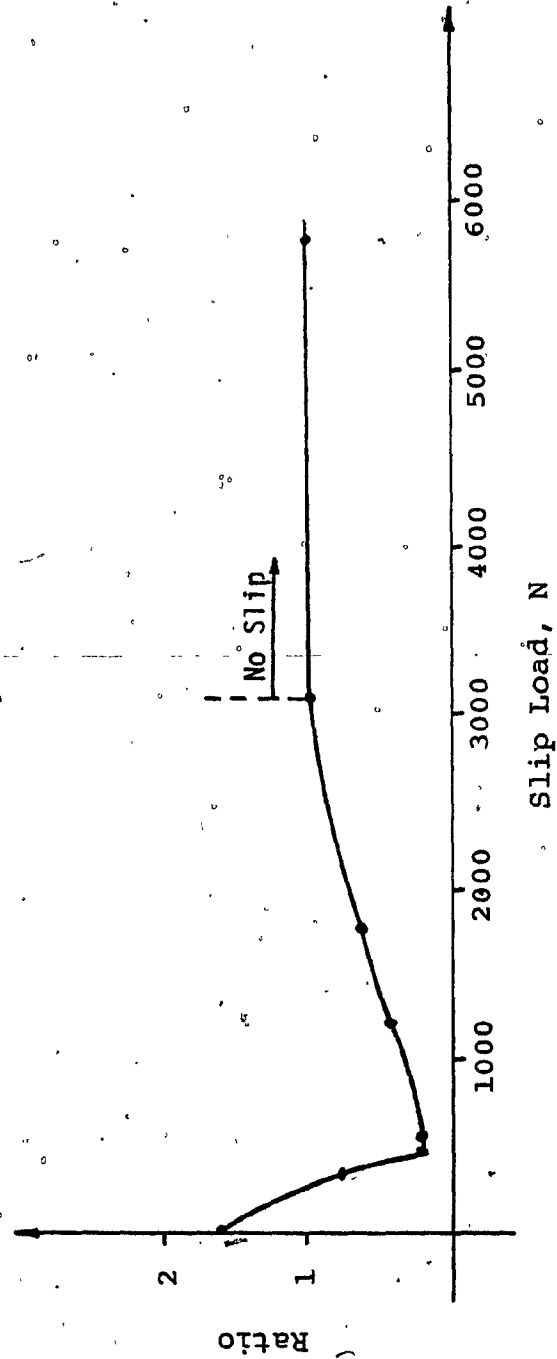Fig. 4.1 - EFFECT OF SLIP LOAD ON DEFLECTIONS

Fig. 4.2 - RATIO OF DEFLECTION OF JOINTED WALL TO MONOLITHIC WALL
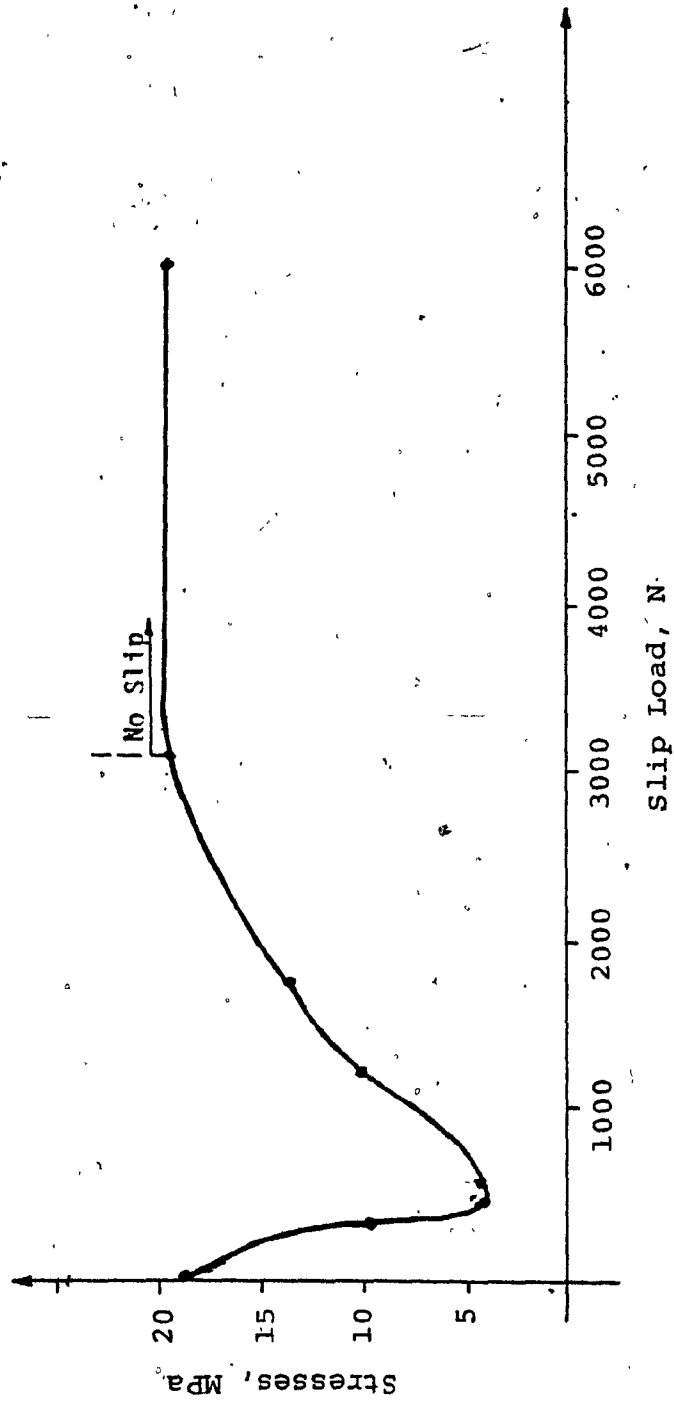
Fig. 4.3 - EFFECT OF SLIP LOAD ON STRESSES

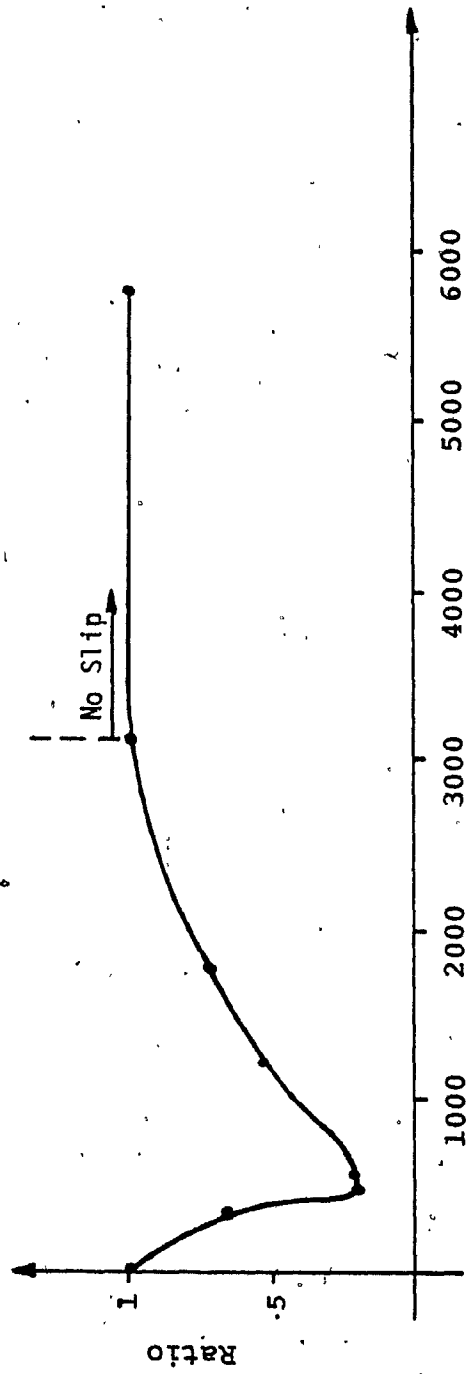Fig. 4.4 - RATIO OF STRESS IN JOINTED WALL TO MONOLITHIC WALL
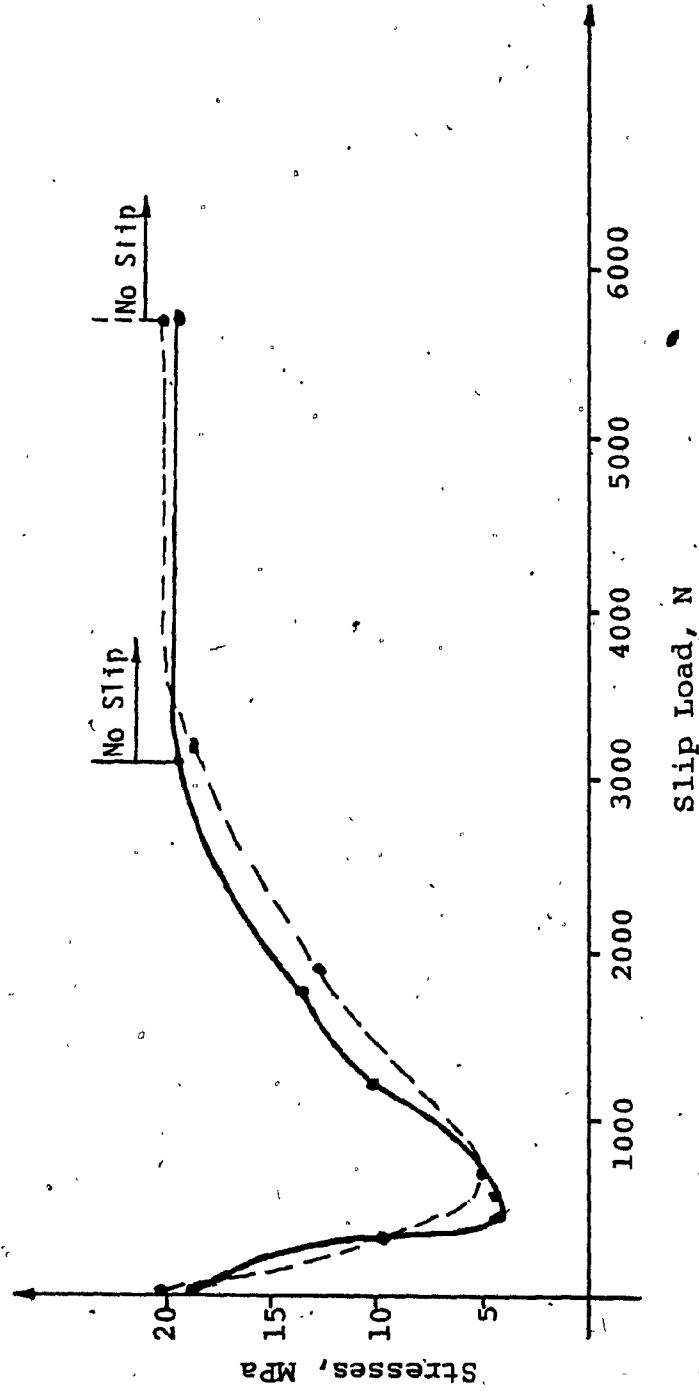
Fig. 4.5 - COMPARISON BETWEEN THE STRESSES OF THE
THEORETICAL AND THE EXPERIMENTAL RESULTS
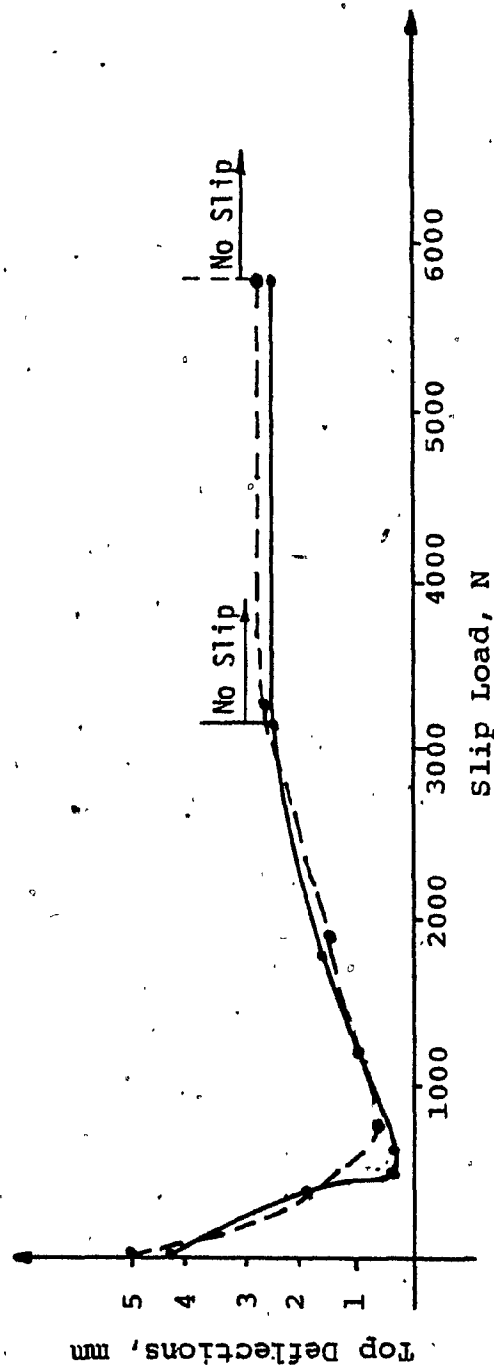
Fig. 4.6 - COMPARISON BETWEEN THE DEFLECTIONS OF THE THEORETICAL AND THE EXPERIMENTAL RESULTS

how limited slip bolted joints can improve the seismic response of shear walls.

The following approximate analysis gives an estimate of the optimum slip load based on limiting stress level.

For exact optimum solution a full nonlinear analysis is required.

4.7.1 Point Load at the Top

The two heavy channel shapes beams on top of the model will be treated as a point mass at the top.

Consider the simple cantilever wall (Fig. 4.7), split vertically, subjected to a point load, P, at the top. The stress distribution across the base of each wall is made up of axial stress and bending stress:

$$\text{axial stress} \qquad \sigma_a = \frac{R}{A}$$

$$\text{bending stress} \qquad \sigma_b = \frac{M}{S}$$

where $A = \frac{bt}{2}$, cross section area of each wall and

$S = \frac{b^2 t}{6}$, the elastic section modulus.

Maximum stress $\sigma_u = \sigma_a + \sigma_b$, is the summation of axial and bending stress.

The strain energy due to bending and axial forces are:

$$\text{due to bending} \qquad U_b = \frac{V \sigma_b^2}{9E} \qquad\qquad (4.4)$$

$$\text{due to axial} \qquad U_a = \frac{V \sigma_a^2}{3E} \qquad\qquad (4.5)$$

where, $V = Ah$, is the volume of each wall and q, is the uniform shear flux causing slipping along the joints between the walls.

The work done against friction, i.e. the energy dissipated in the joints is given by:

$$W_f = 2 \int_o^h q \int_x^h \frac{(\sigma_b - \sigma_a)}{E} \frac{x}{h} \, dx \, dx$$

$$W_f = \frac{2}{3} \frac{q}{E} (\sigma_b - \sigma_a) h^2 \tag{4.6}$$

The total strain energy is equal to:

$$U_a + U_b = \frac{V}{9E} (\sigma_b^2 + 3\sigma_a^2)$$

But:

$$\sigma_b = \sigma_u - \sigma_a$$

So:

$$U_b + U_a = \frac{V}{9E} (\sigma_u^2 - 2\sigma_u \sigma_a + 4\sigma_a^2) \tag{4.7}$$

Knowing $qh = \sigma_a A$, the work done against friction (energy dissipated) is:

$$W_f = \frac{2}{3} \frac{V}{E} (\sigma_u - 2\sigma_a)\sigma_a \tag{4.8}$$

Equations (4.7) and (4.8) are maximum for a limiting stress, σ_u, when:

$$\sigma_a = \frac{\sigma_u}{4} = \frac{\sigma_b}{3} \tag{4.9}$$

For the optimum condition the total strain energy and the energy dissipated are:

strain energy: $U_b + U_a = \frac{1}{12} \frac{V}{E} \sigma_u^2$ $\tag{4.10}$

energy dissipated: $\quad W_f = \frac{1}{12} \quad \frac{V}{E} \cdot \sigma_u^2$ (4.11)

This demonstrates that, the total elastic strain energy is dissipated by friction in a quarter cycle.

By using $qh = A \sigma_a$, the slip flux is :

$$q = \frac{1}{4} \quad \frac{A}{h} \quad \sigma_u$$ (4.12)

so, the slip load per storey is:

$$P_s = \frac{1}{4} \quad \frac{A}{n} \quad \sigma_u$$ (4.13)

where, n is the number of floors.

The moment is given by:

$$Ph = 2\sigma_b \quad \frac{Ab}{6} + \sigma_a \, A \, b = Ab\left(\frac{1}{3} \quad \sigma_b + \sigma_a\right)$$

$$Ph = \frac{1}{2} Ab \, \sigma_u$$

The maximum axial force is:

$$P_{max} = \frac{b}{2} \quad \frac{A}{h} \quad \sigma_u = 2bq$$ (4.14)

load at first slip is:

$$P_e = \frac{4}{3} \quad bq$$ (4.15)

so:

$$\frac{P_{max}}{P_e} = \frac{3}{2}$$ (4.16)

4.7.2 Typical Approximate Design

In order to compare the optimum slip load by approximate analysis to those in the tests, the optimum slip load P, for the lowest stress value from the Figs. 3.4 and 3.5 becomes

$$P_s = \frac{1}{4} \frac{A}{n} \sigma_u$$

$$\sigma_u = 4.7 \qquad \text{MPa, lowest stress value from Fig. 3.4}$$

$$A = 2(2 \times .635 \times 610) \times 2 = 1550 \text{ mm}$$

$$P_s = \frac{1}{4n} (A \sigma_u)$$

$$P_s = \frac{1}{4 \times 3} (1550 \times 4.7) = 607 \text{ N}$$

This is in close agreement with the value established for the optimum slip load.
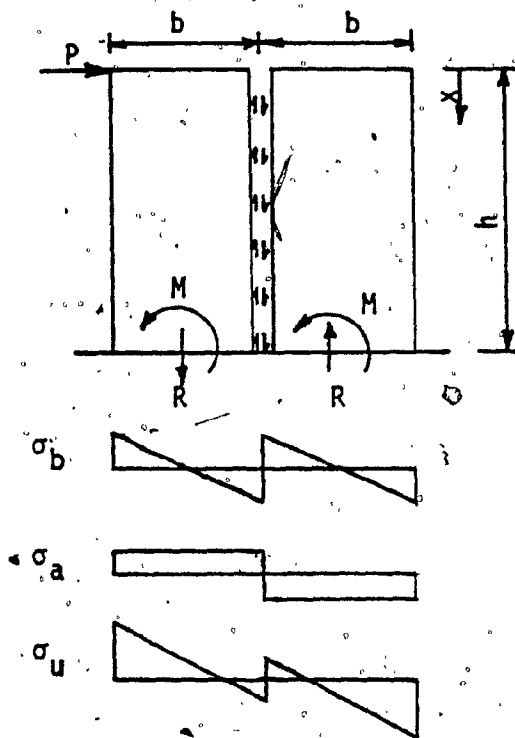
Fig. 4.7 - Coupled Walls (Friction joint)

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

Behavior of structures due to seismic forces is receiving more attention not only to reduce the risk to life but to reduce the secondary damage and economic losses.

It is costly to make tall shear walls strong enough to resist high seismic forces; and the alternative of energy dissipating mechanisms can be economically attractive. In this study the use of these joints in concrete shear walls (cast in place shear walls and precast panel wall systems) have been reviewed and it was concluded that the limited slip bolted joint provides one of the most suitable solutions. In order to understand the behavior of these joints as a source of energy dissipation an experimental study was conducted.

A stapled assembly of sandwich panels with aluminium extrusions incorporating LSB joints simulated a coupled wall. In order to increase the period of vibration and also to avoid any other modes than the first mode a mass was added to the top of the model. To optimize the response of the structure the slip load was varied to give the minimum stress level when the table was resonant with the model, and the table had a predetermined maximum acceleration held constant for all the tests.

The experimental results were compared with those of a nonlinear time/history dynamic analysis using the program "Drain-2D".

The input motion used for the program was the same as that of the tests.

An approximate analysis to find the optimum slip load based on energy method has shown a good agreement with the results obtained and it was concluded that:

1) Comparison of the response of optimum LSB jointed walls with monolithic and isolated walls has shown the efficiency of these joints in improving the seismic response of structures.

2) Comparison between the results found in the tests and the computer analysis using the frame analogy method has shown a good agreement.

3) The simplified analysis to establish the value of the slip load for maximum energy dissipated with a given maximum stress level, has been shown to be in good agreement with the test and computer analysis.

5.2 Recommendations for Future Studies

1) In the present study the optimization was for a specific model with a specific input. Tests with different rigidities, masses, heights, and time histories are required to provide fuller information.

2) Optimum slip loads for typical random earthquake ground motion will differ from that established in this study, and will require further study.

3) If possible, testing of a full scale structure would lead to more realistic results.

4) Effect of and control of torsional oscillations should be studied.

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