SEISMIC BEHAVIOR AND DESIGN OF PRECAST FACADES/CLADDINGS & CONNECTIONS IN LOW/MEDIUM-RISE BUILDINGS

FINAL TECHNICAL REPORT

BY

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Seismic behavior and design of heavy facades/claddings and connections in buildings has been investigated, and unique cyclic racking tests of representative precast concrete facade/cladding panels and connections have been carried out. Test results consist of cyclic load-displacement curves; time-history plots of loads, displacements, accelerations, etc., during each test; analysis of peak response quantities, e.g., displacements and load-levels reached; estimated rigidities of the cladding panel-connection assembly at increasing levels of peak displacements of block cycles; as well as the relationship between drift levels and behavior of cladding panel-connection assemblies. Dynamic testing of a representative reduced scale three dimensional model two story steel-framed building structure with and without precast concrete cladding panels, was carried out. Results provide quantitative experimental data on the earthquake resistance and stiffness of cladding connections and the overall seismic behavior of cladding connection assemblies.
SUMMARY

Seismic behavior and design of heavy facades/claddings and connections in buildings has been investigated, and unique cyclic racking tests of representative precast concrete facade/cladding panels and connections have been carried out. During the first major phase of the research project current practices for design and detailing of heavy facade/claddings and their connections to supporting structural systems, were evaluated. In consultation with practicing architects, engineers, researchers and facade/cladding manufacturers, state-of-the-art data for facade/cladding design, detailing and erection practices was compiled. Available data on the performance of building facade/cladding during previous destructive earthquakes including the recent Mexico City Earthquake of September 1985 was evaluated. Analytical and experimental techniques of modeling the seismic behavior of heavy precast concrete facade/cladding panels and connections have been investigated. The role of modern testing methodology in assessing the seismic behavior of building facades/claddings and connections has been evaluated. Pilot static tests of typical ductile (push-pull) cladding connections were carried out to investigate the strength and behavior of these connections. Cyclic in-plane racking test of a full-size precast concrete cladding panel with bearing connections at the bottom and ductile (push-pull) connections at the top, representative of California current practices, has been carried out. Test results consist of cyclic load-displacement curves; time-history plots of loads, displacements, accelerations, etc., during each test; analysis of peak response quantities, e.g., displacements and load-levels reached; estimated rigidities of the cladding panel-connection assembly at increasing levels of peak displacements of block cycles; as well as the relationship between drift levels and behavior of cladding panel-connection assemblies. Dynamic testing of a representative reduced scale three dimensional model two story steel-framed building structure with and without precast concrete cladding panels, was carried out. Results provide quantitative experimental data on the earthquake resistance and stiffness of cladding connections and the overall seismic behavior of cladding connection assemblies. The test results obtained will help develop improved and more realistic analytical modeling of building structural systems interacting with heavy facades/cladding and connection systems in low/medium-rise buildings during earthquakes.
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CHAPTER 1: INTRODUCTION

This report documents results of a research program carried out to investigate the behavior of heavy facades/claddings and connections in buildings during earthquakes.

The widespread use of heavy facades and claddings in a broad class of buildings in seismic zones, and the potential life-hazards and significant economic losses posed by damage and/or collapse of such heavy exterior finish systems warrants a systematic and thorough examination of the behavior of heavy facades and claddings during earthquakes.

The overall nature and scope of the problem is further evidenced by available observed damage data on the behavior of exterior facade/cladding enclosure systems in buildings during previous earthquakes, e.g., Anchorage, Alaska-1964, San Fernando, California-1971, Miyagi-Ken-Oki, Japan-1978, Mexico City, Mexico-1985, and Whittier-Narrows, California-1987.

A study of the limited available observed damage data clearly shows that mitigation of earthquake damage of building facades/claddings is a very important issue because of the potential hazard to public and significant economic losses posed by such non-structural damage in buildings during earthquakes.

The importance of mitigation of earthquake damage of exterior architectural components, e.g., facades/claddings in buildings was also highlighted at the EERI/NSF workshop (40) on non-structural issues, to attempt to define practical research needs and further research work.

Furthermore, heavy facades and cladding can have significant influence on the overall lateral stiffness of buildings and thus alter the fundamental dynamic properties, e.g., natural frequencies, and also damping, and hence the response and behavior of the overall building system during earthquakes.

It is only recently that efforts have been directed to developing a better understanding of behavior of claddings and connections during earthquakes.

The general lack of an adequate base of test data on the static and cyclic behavior of building facades/claddings and connections, necessitates that testing be carried out to provide quantitative results on the strength and cyclic behavior of typical building facades/claddings and connections, including thresholds of damage, as well as their fundamental characteristics, e.g., natural frequencies, damping, etc.

It is also necessary to document and evaluate the effectiveness of the applicable design provisions of the regulatory standards, e.g., Uniform Building Code (86), ATC 3-06 (7), SEAOC (133), State of California (101), Tri-Services Manual (139) and the recently developed NEHRP Guidelines (28), through correlation with test results and available field data.

1 Numbers in parenthesis refer to Bibliography on page 71.
CHAPTER 2: BUILDING FACADES/CLADDINGS

2.1 BACKGROUND

In general, facades/claddings are regarded as a means of enclosing a building structure by attachment of enclosure material assemblies, capable of spanning between supporting points, on the exterior face of a building. The sizes of the cladding components are based in most part on their ability to resist lateral loads (e.g., wind and earthquakes) acting on the building, and then transfer those loads safely to the building.

The function of building facades/claddings may be described as follows, to provide:

a. Building envelope that protects the interior of the building from all climatic conditions and maintain a comfortable thermal environment.

b. Acoustic insulation that protects the occupants from noise pollution.

c. Fire resistance.

d. Solar protection and possibly reduce the energy demand of HVAC systems.

e. Enhancement to building's external appearance.

Photographs (Figures 1-9) show the many different facade/cladding types, their configurations, materials and exterior finishes in use in low- and medium-rise buildings on the West Coast.
Figure 1  Precast Cladding - Medium-Rise Building
Los Angeles, California

Figure 2  Precast Cladding - Medium-Rise Building
Los Angeles, California
Figure 3  Precast Cladding - Checker-Board Pattern
Medium-Rise Building - San Jose, California

Figure 4  Curtain-Wall Facade - High-Rise Buildings
Downtown, Los Angeles, California
Figure 5  Facade/Cladding Elevation - Medium-Rise Building
Downtown, Los Angeles, California

Figure 6  Spandrel Cladding/Facades - Medium-Rise Building
Downtown, Los Angeles, California
Figure 7  Precast Cladding (Window-Wall Units) - Medium-Rise Building Downtown, Los Angeles, California
Figure 8  Close-up Detail - Precast Cladding (Window-Wall Units) - Medium-Rise Building, Downtown, Los Angeles, California

Figure 9  Precast Cladding - Spandrel - Panels & Column-Cover-Panels, Medium-Rise Building, Downtown, Los Angeles, California
2.2 CLASSIFICATION OF BUILDING FACADE/CLADDING SYSTEMS

For this research report, facades and claddings fastened to moment-resisting frame building systems may be classified as follows:

<table>
<thead>
<tr>
<th>FACADE/CLADDING TYPE</th>
<th>CONFIGURATION</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>WINDOW-WALL PANELS</td>
</tr>
<tr>
<td>I. Precast Concrete Cladding</td>
<td>□</td>
</tr>
<tr>
<td>II. Glass Fiber Reinforced Cement (GFRC) Cladding</td>
<td>□</td>
</tr>
<tr>
<td>III. Masonry Veneer Facades on Framed-Backing</td>
<td>□</td>
</tr>
<tr>
<td>IV. Stone/Granite/Marble Facades on Framed-Backing</td>
<td>□</td>
</tr>
</tbody>
</table>

The above list is not intended to be complete and only represents a partial summary of representative facade and cladding types that should be considered.
2.3 DESIGN ISSUES

Development of facade/cladding systems in buildings in seismic zones requires the consideration of the following design issues:

- **Facade/Cladding Component Issues**
  
  Under this category the following should be considered:

  (i) **Materials**

  From the point of view of earthquake resistance of facades/claddings, the following material issues should be considered in addition to the general considerations of appearance, durability and weather-staining:

  - Mass Properties
  - Strength and Deformation Properties

  (ii) **Geometry and Configuration**

  Important issues under this category are:

  - Shape and Proportions of precast facade/cladding components, e.g., solid shapes, open vs. closed shapes and their combination thereof to provide desired facade/cladding elevations.
  
  - Size of precast facade/cladding components, e.g., length, width, thickness, etc.

- **Connections - Design Issues**

  Important connection design issues are:

  - Types of connections with respect to number, types and methods of load transfer or accommodation of movement/deformation.
  
  - Location of connections.
  
  - Connections between precast facade/cladding components and supporting structural system.
  
  - Connections between precast facade/cladding components.

- **Supporting Structural System - Design Issues**

  The important issues under this category may be summarized as follows:

  Gravity Loads - Supporting structural system must safely carry the weights of the precast facade/cladding components in addition to
usual dead and live loads, through the connections between the precast facade/cladding components and the supporting structure.

Lateral Loads (Wind, Earthquakes) - Supporting structural system must safely resist the effects of lateral loads, e.g., wind and earthquake loading, transmitted through the connections between the facade/cladding components and the supporting structure.

The interrelationship of the above design issues is graphically illustrated in Figure 10.
Figure 10: Facade/Cladding Design Issues and Inter-relationships
CHAPTER 3: SCOPE AND OBJECTIVES

The main focus of this research program is to analytically and experimentally investigate the seismic behavior and design of heavy facades/claddings and their connections in low/medium-rise buildings.

The general objective of this research program is to document and evaluate applicable current provisions of the Uniform Building Code (86) and other regulatory standards, e.g., State of California Title 21 and Title 24 (101), ATC 3-06 (7), SEAOC (131), Tri-Services Manual (139), NEHRP Guidelines (28), and current practices governing the design, detailing and installation of heavy facades/claddings and their connections in low and medium-rise buildings with different framing systems.

In light of the diverse range of facade/cladding components and connections in use in low/medium rise buildings in seismic zones across the U.S., it was decided to focus on investigating the seismic behavior and design of the following exterior finish systems representative of practices in California and other western states.

I. Precast Concrete Cladding Panels Attached to Moment-Resisting Rigid Frame Building Systems

II. Brick Veneer/Granite/Marble Facades on Framed Backing Attached to Moment-Resisting Rigid Frame Building Systems

It should be noted that a significant percentage of exterior building facades/claddings in California, are of the types outlined above.

Upon further consideration it was further decided to focus attention only on the study of Precast Concrete Cladding Panels and their attachments to steel-framed building systems, at this time.
CHAPTER 4: LITERATURE REVIEW

A comprehensive survey of pertinent literature was conducted. The results of this survey are presented in the form of an extensive bibliography (p.71) which provides an exhaustive source of information on a broad range of issues governing behavior, analysis and design of heavy facades/claddings and connections in buildings in seismic zones.

McCue, et al. (93) reported the results of an 'Enclosure Wall - Case Study' as an application of the conceptualized behavior models developed to investigate interaction of building components during earthquakes.

Sack, et al. (118) reported the first detailed investigation of the seismic response of precast curtain-walls in high-rise buildings. This research involved both analytical modeling of precast curtain-wall panels and their connections; as well as testing of curtain-walls and their connections.

Goodno, et al. (66), (67), (68), (69), (102), reported results of investigations of seismic response of glass curtain-walls as well as precast concrete cladding; cladding-structure interaction, analytical modeling for investigating the stiffening effects of cladding on the seismic response of buildings, as well as testing of cladding connections to investigate their behavior.

Wang (147), (148) reported the results of large-scale testing of precast cladding attached to a Full-Scale Steel Test Frame carried out under a U.S.-Japan Cooperative Research Project.
CHAPTER 5: FACADE/CLADDING PERFORMANCE DURING PREVIOUS EARTHQUAKES

In the initial phases of this research project, sincere efforts were made to systematically document the available data on observed performance of non-structural facades/claddings in buildings during previous earthquakes.

The first attempt to systematically document non-structural damage during earthquakes was reported by Ayres, et al. (12) for documenting the non-structural building damage caused by the Anchorage, Alaska, earthquake of 1964. Even though this was an excellent start, no consistent coordinated efforts have since been made to document non-structural building damage in general and facade/cladding damage in particular, during earthquakes since then.

Selected highlights of building facade/cladding performance and damage during the previous earthquakes are presented below as follows:

Table I: Anchorage, Alaska, Earthquake of 1964
Table II: San Fernando, California, Earthquake of 1971
Table III: Miyagi-Ken-Oki, Japan, Earthquake of 1978
Table IV: Mexico City, Mexico, Earthquake of 1985
<table>
<thead>
<tr>
<th>BUILDING NAME</th>
<th>NO. OF STORIES</th>
<th>MATERIAL</th>
<th>FACADE TYPE AND CONNECTION DETAILS</th>
<th>FACADE/CLADDING DAMAGE</th>
<th>SOURCE</th>
</tr>
</thead>
<tbody>
<tr>
<td>J.C. Penney Building</td>
<td>5</td>
<td>Reinforced Concrete</td>
<td>Heavy Facades Precast Concrete Panels</td>
<td>This building suffered catastrophic structural damage as well as non-structural facade damage due to severe torsional displacements caused by the center of rigidity being far removed from the center of mass. The heavy precast concrete facades on the north and east walls contributed to the development of torsional forces and as the structural system failed and became more flexible, the stiffness of the precast facade panels themselves contributed to the failures of the supporting bracket connections. Most of the four inch thick precast panels in the north wall were shaken loose and fell to the street below. Many of the supporting brackets were torn out of floor slabs and were still found to be attached to the backs of facade panels that fell to the street below. Two people were killed when the heavy facade panels fell onto parked cars. Figures 11, 12</td>
<td>Ref.[12]</td>
</tr>
</tbody>
</table>
### Table IA: Facade/Cladding Performance During the Anchorage, Alaska, Earthquake of 1964

<table>
<thead>
<tr>
<th>Building Name</th>
<th>No. of Stories</th>
<th>Material</th>
<th>Facade Type and Connection Details</th>
<th>Facade/Cladding Performance and Damage</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>First Federal Savings</td>
<td>3</td>
<td>Steel-Framed Office Building 50’x130’ in plan.</td>
<td>Variety of exterior wall materials. Glass-Spandrel curtain walls - East, South &amp; portion of West facades.</td>
<td>The curtain walls suffered only minor damage during this earthquake because of compatibility between the flexible curtain wall and the flexible steel frame of the building. One brick panel on the east facade collapsed and the other brick panels on the east and south sides were severely damaged. Rigid non-structural facade were not compatible with the flexible structural frame and therefore the rigid facade suffered extensive damage. The panel on the south facade could not cope with the movements of the steel frame, and was severely damaged. Figures 13, 14</td>
<td>Ref.[12]</td>
</tr>
</tbody>
</table>

**Notes:**
- Lateral force resistance in the N-S direction (long direction) supposed to be provided by a reinforced block wall in the west face and two brick panels in the east face.
- Lateral force resistance in the east-west direction (narrow direction) provided by a reinforced block wall at the north end and a brick panel and an x-braced steel bent in the south face.
SUMMARY OF BUILDING FACADE DAMAGE - ANCHORAGE, ALASKA
EARTHQUAKE OF 1964 [Source Ref. 12]

"1. Heavy precast-concrete panels that were attached to the building frame by clip angles and inserts collapsed.

2. Concrete-masonry-units filler walls were badly cracked and in some instances they damaged the surrounding structural frame.

3. Brick veneers, attached to flexible steel frames without backing or with insufficient backing, cracked and in some instances collapsed. Some stone and brick veneers collapsed where they were improperly tied to concrete walls.

4. Curtain Walls sustained very little damage, except in the vicinity of structural failures. Some mounting brackets broke or pulled loose their concrete inserts at the floor slabs.

5. Glass-block panels were practically undamaged.

6. Window-glass was damaged where adjacent structural elements failed or sustained excessive deflections. Where mounts were rigid and mullions were weak, large panels of glass in storefronts were broken. Some glass panels in curtain walls were damaged when flexible mountings worked loose."
<table>
<thead>
<tr>
<th>BUILDING NAME</th>
<th>Medical Treatment and Care Unit Hospital</th>
</tr>
</thead>
<tbody>
<tr>
<td>NO. OF STORIES</td>
<td>5</td>
</tr>
<tr>
<td>MATERIAL</td>
<td>Reinforced Concrete</td>
</tr>
<tr>
<td>LATERAL FORCE RESISTING SYSTEM</td>
<td>Basic Framing scheme is a two-way flat slab reinforced concrete system supported either on tied or spiral columns. The lateral force resisting system consists of shear walls above the second floor and moment resisting frames in the lower two stories.</td>
</tr>
<tr>
<td>FACADE TYPE AND CONNECTION DETAILS</td>
<td>Precast Concrete Facia Elements</td>
</tr>
<tr>
<td>FACADE/CLADDING PERFORMANCE AND DAMAGE</td>
<td>Many precast concrete facia elements were dislodged. Masonry veneered walls fell away from the building due to earthquake movements. Connections anchoring the concrete failed.</td>
</tr>
<tr>
<td>SOURCE</td>
<td>Ref. [140]</td>
</tr>
</tbody>
</table>

**TABLE II**

**FACADE/CLADDING PERFORMANCE DURING THE SAN FERNANDO, CALIFORNIA EARTHQUAKE OF 1971** [Ref. 139]
### TABLE III  FACADE/CLADDING PERFORMANCE DURING THE MIYAGI-KEN-OKI, JAPAN, EARTHQUAKE OF 1978 [REF. 39]

<table>
<thead>
<tr>
<th>BUILDING NAME</th>
<th>NO. OF STORIES</th>
<th>MATERIAL</th>
<th>FACADE TYPE AND CONNECTION DETAILS</th>
<th>FACADE/CLADDING PERFORMANCE AND DAMAGE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sasaki Building</td>
<td>4</td>
<td>Steel Framed Building</td>
<td>Precast Concrete Curtain Walls</td>
<td>Catastrophic collapse of precast concrete curtain walls. The precast concrete cladding panels broke loose from the building exterior and fell crashing to the ground below onto parked cars. Figure 16</td>
</tr>
<tr>
<td>Izumi City, Japan</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Ref. [39]
### Table IV  Facade/Cladding Performance During the Mexico City, Mexico Earthquake of 1985 *

<table>
<thead>
<tr>
<th>Facade/Cladding Type and Connections</th>
<th>Building Type</th>
<th>Material and Structural System</th>
<th>Facade/Cladding Performance and Damage</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Precast Concrete Cladding Panels in Pino Suarez Building</td>
<td>Medium-Rise Building</td>
<td>Steel Framed Building with moment-resisting frames and braced frame system</td>
<td>Heavy Precast Concrete Cladding attached to this medium-rise steel framed building were subjected to large drift incursions during this earthquake. These large drift levels were responsible for damage of the precast panels which consisted of relative shifting of panels by 3-4 inches. Figures 17, 18.</td>
<td>Ref.[41][42]</td>
</tr>
<tr>
<td>Light Metal-Glass Curtain Walls</td>
<td>Medium-Rise Buildings</td>
<td>Moment-Resisting Concrete Framed Systems</td>
<td>Metal-Glass Curtain Walls in medium-rise buildings suffered moderate levels of damage due to very large distortions (drifts) induced by this earthquake.</td>
<td></td>
</tr>
<tr>
<td>Masonry Infill Facades</td>
<td>Medium-Rise Buildings</td>
<td>Reinforced Concrete Buildings with moment-resisting frames</td>
<td>Masonry Infill facades in medium-rise buildings with reinforced concrete moment-resisting frames, suffered extensive damage during this earthquake. Masonry Infill facades also provided initial lateral force resistance and may have contributed to the survival of many medium-rise buildings during this earthquake. Figures 19-20.</td>
<td></td>
</tr>
</tbody>
</table>

* This brief summary of facade/cladding performance during the Mexico City Earthquake of 1985 is based on observed damage data available to-date.
Figure 11  Collapsed Precast Concrete Facade Panels
J. C. Penney Building
Anchorage, Alaska Earthquake of 1964 (Ref.12)

Figure 12  Collapsed Precast Concrete Facade Panels
J. C. Penney Building
Anchorage, Alaska Earthquake of 1964 (Ref.12)
Figure 13 Facade Damage
First Federal Savings and Loan Building
Anchorage, Alaska Earthquake of 1964 (Ref.12)

Figure 14 Facade Damage
First Federal Savings and Loan Building
Anchorage, Alaska Earthquake of 1964 (Ref.12)
Figure 15 Failure of Precast Concrete Wall Panels
San Fernando, California Earthquake; 1971 (Ref.139)

Figure 16 Collapse of Precast Concrete Curtain Walls
Miyagi-Ken-Oki, Japan Earthquake, 1978 (Ref.39)
Figure 17 Pino-Suarez Building, Mexico City
Damaged Precast Concrete Cladding Already
Removed, Mexico City, Mexico Earthquake of 1985

Figure 18 Pino-Suarez Building, Mexico City
Damaged Precast Concrete Cladding Already
Removed, Mexico City, Mexico Earthquake of 1985
Figure 19  Masonry Infill Facade Damage
Medium-Rise Building With Reinforced Concrete Moment-Resisting Frames,
Mexico City, Mexico Earthquake of 1985

Figure 20  Masonry Infill Facade Damage, Medium-Rise Building
With Reinforced Concrete Moment-Resisting Frames,
Mexico City, Mexico Earthquake of 1985
CHAPTER 6: SEISMIC DESIGN CODES AND REGULATIONS

The provisions of the following codes and regulatory standards governing the seismic design and detailing of facades/claddings and their connections were reviewed:

- ATC 03-6
- UBC
- Tri-Services Manual
- SEAOC
- OSA - State of California
- NEHRP

A summary of the applicable code provisions is presented in Tables V-A and V-B.
## Code Provisions for Seismic Design of Non-Structural Facade/Cladding Panels and Connections

<table>
<thead>
<tr>
<th>FACADE/CLADDING PANELS AND CONNECTIONS</th>
<th>UNIFORM BUILDING CODE 1985 EDITION</th>
<th>PRESTRESSED CONCRETE INSTITUTE - 1977 EDITION</th>
<th>APPLIED TECHNOLOGY COUNCIL ATC 3-06 - 1978</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1. Lateral Design Force Levels for Facade/Cladding Components</strong></td>
<td><strong>GOVERNING PROVISIONS</strong></td>
<td><strong>GOVERNING PROVISIONS</strong></td>
<td><strong>GOVERNING PROVISIONS</strong></td>
</tr>
<tr>
<td></td>
<td>Sec. 2311 Wind Loads</td>
<td>Sec. 2.3.6 Structural Design Considerations</td>
<td>Sec. 3.7.7 Anchorage of Non-Structural Systems</td>
</tr>
<tr>
<td></td>
<td>Sec. 2312(g) Seismic Forces</td>
<td>Sec. 11.3 Seismic Forces</td>
<td>Sec. 8.2 Architectural Design Requirements</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Sec. 11.4 Design Guidelines for Panels</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( F_p = ZIC_p W_p ) (EQ 12-8)</td>
<td>( F_p = ZIC_p W_p ) (EQ 2-6)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( C_p ) = Table 23-J</td>
<td>( C_p = 0.2 ) for Panels</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( * I_p = Table 23-K )</td>
<td>( C_p = 2.0 ) for Connections</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( K_p ) = Table 23-1</td>
<td>IF ( C_p = 2.0 ) THEN ( I.S = 1.0 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( Z ) = Figures 23-1, 23-2, 23-3</td>
<td>ALL OTHER VALUES TAKEN FROM THE UBC</td>
<td></td>
</tr>
<tr>
<td></td>
<td>* FOR PANELS ( I = 1.0 )</td>
<td>NOTE: EQ 12-6 IS VALID FOR BOTH IN-PLANE AND OUT-OF-PLANE FORCES.</td>
<td></td>
</tr>
<tr>
<td></td>
<td>NOTE: EQ 12-8 IS VALID FOR IN-PLANE AND OUT-OF-PLANE FORCES.</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>2. Loads Due to Volumetric Changes</strong></td>
<td>Sec. 2.3.4 Force Systems</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>EQ: 2-2, 2-3, 2-4, 2-5</td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>3. Loads Due to Shipping and Handling</strong></td>
<td>Sec. 2.3.3 Erection Considerations</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### TABLE V-A
<table>
<thead>
<tr>
<th>FACADE/CLADDING PANELS AND CONNECTIONS</th>
<th>UNIFORM BUILDING CODE 1985 EDITION</th>
<th>PRESTRESSED CONCRETE INSTITUTE - 1977 EDITION</th>
<th>APPLIED TECHNOLOGY COUNCIL ATC 3-06 - 1978</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>4. DRIFT PROVISIONS</strong></td>
<td><strong>GOVERNING PROVISIONS</strong></td>
<td><strong>GOVERNING PROVISIONS</strong></td>
<td><strong>GOVERNING PROVISIONS</strong></td>
</tr>
<tr>
<td>Sec. 2312(h)</td>
<td>Sec. 2.3.6</td>
<td>Sec. 3.8</td>
<td>Sec. 4.6</td>
</tr>
<tr>
<td>INTER-STORY DRIFT =</td>
<td>FOLLOW 1976 UBC</td>
<td>DEFLECTION &amp; DRIFT LIMITS</td>
<td>DRIFT DETERMINATION AND P-Δ EFFECTS</td>
</tr>
</tbody>
</table>
| \[
| \text{INTER-STORY LATERAL DEFLECTION UNDER DESIGN SEISMIC FORCES} \]
| \cdot [1.0/K]                          | DEFLECTION MUST BE LESS THAN:       | \[ \Delta = \delta x_2 - \delta x_1 \]     | \[ \Delta = \delta x_2 - \delta x_1 \]   |
| WHERE                                   | a) 2/K (WIND DRIFT)                  | \[ \delta x_1 = \text{DEFLECTION AT 1st FLOOR} \] | \[ \delta x_1 = \text{DEFLECTION AT 1st FLOOR} \] |
| K...GIVEN BY TABLE 23-I                | b) 3/K (SEISMIC DRIFT)               | \[ \delta x = C_d \delta x_e \text{ (EQ 4-9)} \] | \[ \delta x = C_d \delta x_e \text{ (EQ 4-9)} \] |
| MAX. INTER-STORY DRIFT < 0.005h         | c) 1/4 INCH                          | \[ \delta C_d = \text{DEFLECTION AMPLIFICATION FACTOR...TABLE 3B} \] | \[ \delta C_d = \text{DEFLECTION AMPLIFICATION FACTOR...TABLE 3B} \] |
| h = STORY HEIGHT                        | WHICHEVER IS GREATER.                | \[ \delta x_e = \text{DEFLECTIONS DETERMINED BY ELASTIC ANALYSIS} \] | \[ \delta x_e = \text{DEFLECTIONS DETERMINED BY ELASTIC ANALYSIS} \] |
|                                        | K = HORIZONTAL FORCE FACTOR \text{...TABLE 11-1} | | |
| Sec. 2312(j)3C                         | Sec. 11.3                            | Sec. 2.5                                     | SEISMIC FORCES SAME AS FOR DESIGN OF PANEL |
| \[ F_{\text{connection}} = \frac{1}{3} F_p \] | SEISMIC FORCES                       | ANALYSIS AND DESIGN OF CONNECTIONS          | MOVEMENT OF PANEL SHALL ACCOMMODATE THE STORY DRIFT CALCULATED USING SECTION 4.6 |
| \[ F_{\text{bolt or weld}} = 4 F_p \]  | \[ F_p = 2Z W_p \text{ (EQ 11-3)} \] | CONNECTION DETAILS FOR NON-LOAD BEARING PANELS | |
| RELATIVE MOVEMENT OF THE CONNECTIONS   | \[ \Delta = oX_2 - oX_1 \]           | | |
| \[ oX_1 = \text{DEFLECTION AT 1st FLOOR} \] | \[ oX = C d oX_e \text{ (EQ 4-9)} \] | | |
| \[ oX_e = \text{DEFLECTIONS DETERMINED BY ELASTIC ANALYSIS} \] | \[ oX = C d oX_e \text{ (EQ 4-9)} \] | | |
| \[ 0X_e = \text{DEFLECTIONS DETERMINED BY ELASTIC ANALYSIS} \] | \[ \delta C_d = \text{DEFLECTION AMPLIFICATION FACTOR...TABLE 3B} \] | | |
| \[ \delta x_e = \text{DEFLECTIONS DETERMINED BY ELASTIC ANALYSIS} \] | \[ \delta x_e = \text{DEFLECTIONS DETERMINED BY ELASTIC ANALYSIS} \] | | |
| \[ \delta x = C_d \delta x_e \text{ (EQ 4-9)} \] | \[ \delta x = C_d \delta x_e \text{ (EQ 4-9)} \] | | |
| \[ \delta C_d = \text{DEFLECTION AMPLIFICATION FACTOR...TABLE 3B} \] | \[ \delta C_d = \text{DEFLECTION AMPLIFICATION FACTOR...TABLE 3B} \] | | |
| \[ \delta x_e = \text{DEFLECTIONS DETERMINED BY ELASTIC ANALYSIS} \] | \[ \delta x_e = \text{DEFLECTIONS DETERMINED BY ELASTIC ANALYSIS} \] | | |
| \[ \delta x_1 = \text{DEFLECTION AT 1st FLOOR} \] | \[ \delta x_1 = \text{DEFLECTION AT 1st FLOOR} \] | | |
| \[ \delta x = C_d \delta x_e \text{ (EQ 4-9)} \] | \[ \delta x = C_d \delta x_e \text{ (EQ 4-9)} \] | | |
| \[ \delta C_d = \text{DEFLECTION AMPLIFICATION FACTOR...TABLE 3B} \] | \[ \delta C_d = \text{DEFLECTION AMPLIFICATION FACTOR...TABLE 3B} \] | | |
| \[ \delta x_e = \text{DEFLECTIONS DETERMINED BY ELASTIC ANALYSIS} \] | \[ \delta x_e = \text{DEFLECTIONS DETERMINED BY ELASTIC ANALYSIS} \] | | |
| NOTE: CONNECTIONS SHALL BE DESIGNED TO PERMIT MOVEMENT IN THE PLANE OF THE PANEL EQUAL TO THE DEFLECTION CALCULATED. | | | |

**Table V-A**
## Code Provisions for Seismic Design of Non-Structural Facade/Cladding Panels and Connections

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>1. LATERAL DESIGN FORCE LEVELS FOR FACADE/CLADDING COMPONENTS</strong></td>
<td><strong>GOVERNING PROVISIONS</strong></td>
<td><strong>GOVERNING PROVISIONS</strong></td>
<td><strong>GOVERNING PROVISIONS</strong></td>
</tr>
<tr>
<td>Section 9-3</td>
<td>Seismic forces</td>
<td>Seismic force ( F_p ) determined similar to UBC</td>
<td>Sec. 8.2.2</td>
</tr>
<tr>
<td>( F_p = ZIC_pW_p ) (Eq 3-8)</td>
<td></td>
<td>( F_p = A_vC_pW_p ) (Eq 8-1)</td>
<td>Seismic force applied to building component at its center of gravity</td>
</tr>
<tr>
<td>( C_p = 0.3 ) [Table 3-4]</td>
<td></td>
<td>( C_p = \text{Seismic coefficient for architectural components given in Table 8-B. Varies from } 0.6-3.0 \times \text{performance factor related to life safety (0.5-1.5)} )</td>
<td></td>
</tr>
<tr>
<td>Special provisions for exterior elements</td>
<td></td>
<td>( A_v = \text{Seismic coefficient representing the effective-peak-velocity-related acceleration per Sec. 1.4} )</td>
<td></td>
</tr>
<tr>
<td>I......importance coefficient same as value used for the building</td>
<td></td>
<td>( P = \text{Performance criteria factor given in Table 8-A} )</td>
<td></td>
</tr>
<tr>
<td>Section 3-3(J)3d</td>
<td>( w_p )......weight of facade/cladding component</td>
<td>( W_c )......weight of building component</td>
<td></td>
</tr>
<tr>
<td>Section 3-3(D)-1</td>
<td>( z )......numerical coefficient related to seismicity of a region.</td>
<td>( W_c )......weight of building component</td>
<td></td>
</tr>
<tr>
<td>Note: Eq 3-8 is valid for in-plane and out-of-plane forces.</td>
<td></td>
<td>Note: The force ( F_p ) shall be applied independently longitudinally (in-plane), laterally (out-of-plane), or vertically in combination with weight of component.</td>
<td></td>
</tr>
<tr>
<td><strong>2. LOADS DUE TO VOLUMETRIC CHANGES</strong></td>
<td>Sec. 3-3(J)3d</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Special provisions for exterior elements...temperature changes</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>3. LOADS DUE TO SHIPPING AND HANDLING</strong></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

**Table V-B**
**CODE PROVISIONS FOR SEISMIC DESIGN OF NON-STRUCTURAL FACADE/CLADDING PANELS AND CONNECTIONS**

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>4. DRIFT PROVISIONS</td>
<td>GOVERNING PROVISIONS</td>
<td>GOVERNING PROVISIONS</td>
<td>GOVERNING PROVISIONS</td>
</tr>
<tr>
<td></td>
<td>SEC. 3-3(H) INTER-STORY DRIFT</td>
<td>MAX INTERSTORY DRIFT ≤ 0.005h</td>
<td>SEC 3.8 DEFLECTION AND DRIFT LIMITS</td>
</tr>
<tr>
<td></td>
<td>INTER-STORY LATERAL DEFLECTION UNDER DESIGN SEISMIC FORCES * (1.0/K)</td>
<td>h = STORY HEIGHT</td>
<td>DESIGN STORY DRIFT Δ ≤ ALLOW. STORY DRIFT Δa</td>
</tr>
<tr>
<td></td>
<td>WHERE 1.0/K &gt; 1.0</td>
<td>MAX INTERSTORY DRIFT ≤ 0.0025h'</td>
<td>TABLE 3-C</td>
</tr>
<tr>
<td></td>
<td>K = NUMERICAL COEFFICIENT GIVEN BY TABLE 3-3</td>
<td>h' = HEAD TO SILL OF GLAZED OPENINGS</td>
<td>ALLOWABLE STORY DRIFT Δa = 0.010-0.015h'sx</td>
</tr>
<tr>
<td></td>
<td>MAX. INTER-STORY DRIFT ≤ 0.005h</td>
<td>h = STORY HEIGHT</td>
<td>BASED ON SEISMIC HAZARD EXPOSURE GROUP</td>
</tr>
<tr>
<td></td>
<td>h = STORY HEIGHT</td>
<td>MAX. INTERSTORY DRIFT ≤ 0.005h'</td>
<td>h'sx = STORY HEIGHT</td>
</tr>
<tr>
<td></td>
<td></td>
<td>h' = HEAD TO SILL OF GLAZED OPENINGS</td>
<td></td>
</tr>
</tbody>
</table>

**TABLE V-B**
# Code Provisions for Seismic Design of Non-Structural Facade/Cladding Panels and Connections

|----------------------------------------|---------------------------------------|-------------------------------------|-------------------------------------|
| 5. PROVISIONS FOR DESIGN OF CONNECTIONS BETWEEN FACADE/CLADDING PANELS AND THE STRUCTURAL FRAME | F_{\text{connection}} = 1.33 F_p  
F_{\text{bolts, welds, inserts}} = 4F_p  
ALLOWABLE RELATIVE MOMENTS OF THE CONNECTIONS & PANEL JOINTS:  
allow < 2 \cdot (\text{WIND DRIFT})  
< 3/K \cdot (\text{SEISMIC DRIFT})  
< 1/2 \text{ INCH} |
| GOVERNING PROVISIONS | SAME AS FOR UBC |
| GOVERNING PROVISIONS | |
| GOVERNING PROVISIONS | SEC. 8.2.3  
EXTERIOR WALL PANEL ATTACHMENT  
CONNECTIONS SHALL HAVE SUFFICIENT DUCTILITY AND PROVIDE ROTATIONAL CAPACITY NEEDED TO ACCOMMODATE THE DESIGN STORY DRIFT DETERMINED BY SEC. 4.6.1.  
FACADE/CLADDING PANELS CONNECTED TO STRUCTURAL FRAMING SYSTEM MUST BE ABLE TO ACCOMMODATE THE DESIGN STORY DRIFT WITHOUT FAILURE. |

**Table V-B**
CHAPTER 7: REVIEW OF CURRENT DESIGN AND CONSTRUCTION PRACTICES

7.1 FACADES/CLADDING PANELS

7.2 CONNECTIONS

A schematic block diagram of the overall design process governing the seismic design and detailing of non-structural facades/cladding components and connections in buildings is presented on p.

Basically, the current facade/cladding and connections design and detailing practices are based on the following:

- **Seismic Design Codes and Regulations**, e.g., UBC, ATC, Tri-Services Manual, SEAOC, OSA, NEHRP
  
  Comparative evaluation of applicable seismic design codes was presented in Chapter 6.

- **Industry Standards and Guidelines**

  Guidelines for design, detailing, production, and erection of precast concrete facade/cladding panels and connections are provided by Prestressed Concrete Institute (106), (107), (108), (109), (110), (124), Precast Product Manufacturers (89) and others (103).

**Current Facade/Cladding Construction Practices**

**GFRC Cladding Panels**

This type of cladding is becoming increasingly popular on the West Coast. Figure 21 shows a GFRC cladding panel fabricated at a precasting plant before being shipped to the construction site.

Figure 22 shows a typical GFRC cladding panel being lifted for shipment at a precasting plant on the West Coast.

**Precast Concrete Spandrel Panels**

This type of facade/cladding is widely used not only on the West Coast but other states as well, in the United States.

Figures 23 and 24 show typical precast concrete spandrel panels being delivered to a construction site in the San Francisco Bay Area. The precast panels already have steel-angle-attachment assemblies embedded in them during the panel fabrication process.

Figure 25 shows typical layout and configuration of precast concrete spandral panels during construction in a low-rise steel-framed building near San Francisco.

Figure 26 shows close-up detail of precast concrete spandrel panels and column-cover-panels during construction.
Figure 27 shows the installation of precast column-cover-panels in progress in a low-rise steel-framed building near San Francisco.

**Precast Concrete Window-Wall Cladding Panels**

Figure 28 shows the installation and connection details of a story-high precast concrete cladding panel in a steel-framed high-rise buildings in San Francisco.

Figure 29 shows the detailing and installation of precast concrete cladding corner units in a steel-framed high-rise building in San Francisco.

**Precast Concrete Facades/Claddings and Connections**

In light of the diverse range of facade/cladding components and connections used in low/medium-rise buildings in seismic zones across the U.S., it was decided to focus on investigating the seismic behavior and design of precast concrete cladding panels attached to rigid-frame building structural systems representative of current practices in the U.S. It was further decided to focus on the investigation of seismic behavior and design of story-high window-wall panel components and connections in buildings with moment-resisting frame structural systems.

**Connections**

A study of the state-of-the-art of seismic design and detailing of cladding connections shows that there are many different types of connections and details in use in different parts of the U.S.

According to current design practice in California and other seismic zones of the U.S., Ref. (89), (93), (106), (53), (124), (125), (103), (108), (109), (110), (147), (148) connections of precast concrete window-wall facade/cladding panels to the building structural frames may be divided into the following categories:

- **Flexible Connection at Top**

  Typically there are two attachment points at top of the cladding panel. These flexible or push-pull connections between the cladding panel and the structural frame are expected to accommodate all possible differential movements including inter-story drifts caused by lateral load, e.g., wind and earthquakes; as well as differential movements due to unbalanced gravity loads, temperature changes, creep and shrinkage.

- **Bearing Connection at Bottom**

  Typically there are two attachment points at the bottom of the cladding panel. These rigid connections at the bottom of cladding panels are designed to provide resistance to gravity and lateral loads, e.g., wind and earthquakes.
In current design practice, it is assumed that cladding contributes only mass to building system. Thus the designer accounts for facade/cladding in the seismic design process by including only the dead weight of cladding panels tributary to building floor under consideration. The total mass distribution in the building, thus obtained is used along with the lateral stiffness of the building to determine fundamental dynamic properties, e.g., modal frequencies and mode shapes, as well as seismic response analysis and design of the building system.

It is also assumed that the flexible lateral connections at top of the cladding panels provide no in-plane earthquake resistance and function only to accommodate differential movements between the facade/cladding panels and building structural frames.
SEISMIC DESIGN PROCESS FOR NON-STRUCTURAL FACADE/CLADDING COMPONENTS AND CONNECTIONS

SCHEMATIC

PRELIMINARY DESIGN

DESIGN EARTHQUAKE FORCES

SEISMIC ANALYSIS OF STRUCTURAL SYSTEM

CHECK SIZES OF STRUCTURAL MEMBERS

INTER-STORY DRIFT

DESIGN NON-STRUCTURAL FACADE/CLADDING COMPONENTS

DESIGN NON-STRUCTURAL FACADE/CLADDING CONNECTIONS

SEISMIC DESIGN COMPLETE

PROCEDURES

I. Establish: 1. Floor Dead & Live Loads
   2. Preliminary Cladding Configuration, Sizes, Loads
   3. Story Heights and Elevations
   4. Design Preliminary Sizes

II. Establish: Seismic Base Shear Distribution of Seismic Forces

III. Perform Seismic Analysis of Structural System
     Determine Member Forces and Deflections

IV. Compile Results of Seismic Analysis of Structural System
    Check Strength Required vs. Strength Provided

V. Compile Deflection Results From Seismic Analysis of Structural System

VI. Design Precast Concrete Panels
    1. Calculate Center of Gravity of Panel
    2. Compute Design Loads
       a. Gravity...Dead & Live Loads
       b. Lateral...Wind & Seismic
       c. Volumetric Changes...Shrinkage, Creep, Temperature
       d. Handling...Stripping, Shipping, Erection
    3. Design Panel Using PCI and ACI Specifications

VII. Choose Type of Connection
    1. Bolted, Welded
    2. Clip Angle
Figure 21 GFRC Cladding Panels During Fabrication at Fabrication Plant

Figure 22 Typical GFRC Cladding Panel Being Lifted for Shipment at Precasting Plant
Figure 23 Typical Precast Concrete Spandrel Panels Being Delivered to Construction Site

Figure 24 Typical Precast Concrete Spandrel Panels Being Delivered to Construction Site
Figure 25  Typical Configuration of Precast Concrete Spandrel Panels in a Low-Rise Steel-Framed Building - During Construction
Figure 26 Close-up of Precast Concrete Spandrel Panels and Column-Cover-Panels - During Construction
Figure 27 Installation of Precast Concrete Column-Cover-Panel in a Low-Rise Steel-Framed Buildings
Figure 28 Installation and Connection Details of a Story-High Precast Connection Cladding Panel in a Steel-Framed High-Rise Building

Figure 29 Installation and Details of Precast Concrete Cladding Corner Units in a Steel-Framed High-Rise Building
CHAPTER 8: TESTING PROGRAM

GENERAL

In light of a general lack of test data on claddings and connections, a testing program was developed and carried out to investigate the behavior of precast concrete cladding panels with threaded-rod flexible lateral connections at top and rigid bearing connections at bottom, representative of design practices on the west coast of the U.S.

8.1 TEST I TESTING OF LATERAL (THREADED-ROD) CONNECTIONS

8.1.1 TEST OBJECTIVE

The objective of these tests was to study the static load-deflection behavior of 5/8 inch diameter threaded rods of different lengths and support conditions representative of those used in precast concrete cladding panels.

8.1.2 DESCRIPTION OF TEST SPECIMEN

Test I specimens consisted of a mock-up assembly of flexible lateral connection at the top part of a precast concrete cladding panel. The mock-up assembly consisted of a block of concrete 4 inches thick, 11 inches high and 40 inches long. Threaded-rods of different lengths, e.g., 4, 6, 8, 10 and 12 inches were connected to the block of concrete by a typical assembly consisting of a steel plate with a hole at the center and a Ferrule insert welded to the back of the plate in addition to four headed studs, as shown in Figures 2, 3 (Appendix A).

8.1.3 TEST SET-UP AND PROCEDURE

The overall test set-up is shown in Figures 1, 4, 5 (Appendix A). Loading was applied by means of a loading structural Tee with a 2-inch diameter hole, with 1/4-inch thick washers and one nut on each side of the stem of the loading Tee. Loading was applied using a Riehle Universal Testing machine, and threaded-rod deflections were measured using dial gages. Each threaded-rod specimen was subjected to statically applied loading and unloading. A uniaxial tensile test of a 5/8-inch diameter threaded-rod was also carried out to investigate the behavior of such a rod in axial tension and establish its fundamental strength and deformation properties.

8.1.4 TEST RESULTS

A summary of results of static tests of threaded-rod lateral connections is presented in Table VI. Typical load-deflection curves for all threaded-rods are presented in Figs. 7 to 11 (Appendix A).

Based on an experimentally obtained uniaxial tensile stress-strain curve for a 5/8 inch diameter threaded-rod (Fig.6
-Appendix A), an analytical model for prediction of the load-deflection relationship for the threaded-rods tested, was developed. A plot of estimated stiffness of threaded-rod specimens at different load levels is presented in Fig. 12 (Appendix A).
## TEST I: STATIC TESTS OF THREADED ROD-TYPE LATERAL CONNECTIONS

<table>
<thead>
<tr>
<th>SPECIMEN NO.</th>
<th>THREADED ROD</th>
<th>MAX. TEST LOAD</th>
<th>MAX. ROD DEFLECTION</th>
<th>MAX. BENDING STRESS IN ROD @ MAX. LOAD: BASED ON ANALYTICAL MODEL KSI</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>LENGTH IN.</td>
<td>DIA. IN.</td>
<td>IN.</td>
<td></td>
</tr>
<tr>
<td>CST-L4</td>
<td>4</td>
<td>0.625</td>
<td>478</td>
<td>0.64</td>
</tr>
<tr>
<td>CST-L4A</td>
<td>4</td>
<td>0.625</td>
<td>415</td>
<td>0.78</td>
</tr>
<tr>
<td>CST-L6</td>
<td>6</td>
<td>0.625</td>
<td>290</td>
<td>0.87</td>
</tr>
<tr>
<td>CST-L6A</td>
<td>6</td>
<td>0.625</td>
<td>290</td>
<td>0.79</td>
</tr>
<tr>
<td>CST-L8</td>
<td>8</td>
<td>0.625</td>
<td>180</td>
<td>1.34</td>
</tr>
<tr>
<td>CST-L8A</td>
<td>8</td>
<td>0.625</td>
<td>178</td>
<td>1.00</td>
</tr>
<tr>
<td>CST-L10</td>
<td>10</td>
<td>0.625</td>
<td>133</td>
<td>0.86</td>
</tr>
<tr>
<td>CST-L10A</td>
<td>10</td>
<td>0.625</td>
<td>145</td>
<td>0.95</td>
</tr>
<tr>
<td>CST-L12</td>
<td>12</td>
<td>0.625</td>
<td>103</td>
<td>1.11</td>
</tr>
<tr>
<td>CST-L12A</td>
<td>12</td>
<td>0.625</td>
<td>104</td>
<td>1.06</td>
</tr>
</tbody>
</table>

**TABLE VI: SUMMARY OF TEST RESULTS**
8.2 TEST II CYCLIC TESTS OF PRECAST CONCRETE CLADDING PANELS AND CONNECTION ASSEMBLY

8.2.1 TEST OBJECTIVE

The objective of Test II was to investigate the in-plane resistance and behavior of full-size precast concrete cladding panels and connections under cyclic displacements of increasing amplitudes and different frequencies.

8.2.2 DESCRIPTION OF TEST SPECIMEN

Test II specimen consisted of a solid precast concrete cladding panel 8' wide x 10' high x 4-1/2" thick, with two threaded-rod lateral connections at top of panel and two bearing connections at the bottom. The bearing connection consists of a steel angle assembly with four 5/8-inch diameter studs welded to back of the angle, and embedded in the cladding panel. Two threaded-rod lengths of 6 and 8 inches were used for Test II.

Figure 30 shows an overall schematic of Test II Precast Cladding Specimen including location of Threaded-Rod Flexible Connections and Rigid Bearing Connections.

Details of cladding cyclic test specimen and top and bottom connections are shown in Figures 1, 2, 3, 4, 5 (Appendix B).

8.2.3 TEST SET-UP AND PROCEDURE

The overall cyclic test set-up for Test II is shown in Figures 1, 4 (Appendix B). The cyclic displacements were applied to the precast cladding specimen through a loading assembly attached to the threaded-rod lateral connections as shown in Figures 2, 4 (Appendix B).

The cladding test specimens were subjected to cyclic racking motions using an MTS electro-hydraulic shaking system located in the High-Bay Laboratory of the School of Architecture. An overview of the dynamic testing instrumentation set-up is presented in the Block Diagram of Figure 31. The cyclic test sequence consisted of block cyclic tests. During each test run frequency was fixed at 0.1 Hz or 0.5 Hz and the test specimen was subjected to five cycles of loading for each peak command displacement starting with 1/4, 3/8, 1/2, 3/4, 1, 1-1/2, 1-3/4, 2, 2-1/2 inches.

A summary of the Cyclic Test Control Parameters is presented in Table VII.

Representative Cyclic Test Data and Cyclic Load-Displacement Curves are presented in Appendix B (Figures 11-15).
8.2.4 TEST RESULTS

Time-History Data for all transducer channels was analyzed and peak-responses were recorded. Representative plots of time-history data for force, displacement and strain are presented in Figures 11-20 (Appendix B). The peak-response data for all cyclic test runs is presented in Tables I-IV (Appendix B).

The observed behavior and fracturing of threaded-rod lateral connection under cyclic displacements just prior to failure is shown in Fig. 32. Graphs of peak lateral-force resistance of threaded-rod lateral connections vs. horizontal displacement (drift) are shown in Figs. 16, 17 (Appendix B). A summary of cyclic test results for the precast cladding specimens with 6-inch and 8-inch long threaded-rod lateral connections is presented in Tables VIII and IX. These tables document not only the peak load and horizontal displacement (drift) levels reached but also present estimates of service load-surcharge to the bearing angle for each of the test runs up to failure. The service-load surcharge is expressed as a percentage of the standard design load both for the bearing connection angles and the headed-studs in the bearing connection. Details of the computation of the service-load surcharge to the bearing connection due to the resistance of the threaded-rod connections are given in Appendix B.
FIG. 30 TEST II
SCHEMATIC OVERVIEW OF PRECAST CLADDING TEST SPECIMEN AND CONNECTIONS
NSF SPONSORED RESEARCH PROJECT
SEISMIC TESTING OF HEAVY PRECAST FACADES/CLADDING PANEL
AND CONNECTIONS
PRINCIPAL INVESTIGATOR: DR. SAT RIHAL, ARCH. ENGR. DEPT.
PREPARED BY: DR. GARY GRANNEMAN, ET/EL DEPT.
DATE: 8/86
CALIFORNIA POLYTECHNIC STATE UNIVERSITY,
SAN LUIS OBISPO, CALIFORNIA 93407

![Block Diagram](image-url)
NSF SPONSORED RESEARCH PROJECT
SEISMIC TESTING OF HEAVY PRECAST FACADES/CLADDING & CONNECTIONS

TABLE VII: SUMMARY OF DYNAMIC TEST CONTROL PARAMETERS

<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>PRECAST CONCRETE CLADDING PANEL WITH THREADED-ROD LATERAL CONNECTIONS @ TOP &amp; BEARING CONNECTIONS @ BOTTOM</th>
<th>FREQUENCY Hz</th>
<th>NO. OF CYCLES</th>
<th>BLOCK CYCLIC TEST COMMAND PEAK DISPLACEMENT OF CYCLES - INCHES</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>SIZE</td>
<td>PANEL THICKNESS INCHES</td>
<td>THREAD-ROD LENGTH INCHES</td>
<td></td>
</tr>
<tr>
<td></td>
<td>FPCRT-L6</td>
<td>8'-0&quot; WIDE X 10'-0&quot; HIGH</td>
<td>4 1/2</td>
<td>6</td>
</tr>
<tr>
<td></td>
<td>FPCRT-L8</td>
<td>8'-0&quot; WIDE X 10'-0&quot; HIGH</td>
<td>4 1/2</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
FIG. 32 TEST II
TYPICAL CYCLIC BEHAVIOR AND FRACTURING
OF THREADED-ROD CONNECTIONS JUST PRIOR
TO FAILURE
## TEST II: IN-PLANE CYCLIC TEST OF PRECAST FACADE/CLADDING PANEL AND CONNECTIONS

### SUMMARY OF TEST RESULTS - ESTIMATE OF LOAD SURCHARGE TO BEARING CONNECTION

#### THREADED ROD - LENGTH = 6 INCHES. TEST FREQUENCY = 0.1 Hz

<table>
<thead>
<tr>
<th>RUN</th>
<th>HORIZONTAL RELATIVE DISPLACEMENT (DRIFT) Δ INCHES</th>
<th>MAX. PEAK LOAD-CELL READING kips</th>
<th>Δ/H</th>
<th>ESTIMATED LOAD SURCHARGE AS % OF STANDARD DESIGN LOAD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TO BEARING CONNECTION</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TO STUDS IN BEARING CONNECTION</td>
</tr>
<tr>
<td>AF 1</td>
<td>0.171</td>
<td>1.075</td>
<td>0.0014</td>
<td>52</td>
</tr>
<tr>
<td>AF 3</td>
<td>0.374</td>
<td>1.466</td>
<td>0.0031</td>
<td>70</td>
</tr>
<tr>
<td>AF 4</td>
<td>0.591</td>
<td>1.661</td>
<td>0.0049</td>
<td>80</td>
</tr>
<tr>
<td>AF 6</td>
<td>0.811</td>
<td>1.759</td>
<td>0.0068</td>
<td>84</td>
</tr>
</tbody>
</table>

#### THREADED ROD - LENGTH = 6 INCHES. TEST FREQUENCY = 0.5 Hz

<table>
<thead>
<tr>
<th>RUN</th>
<th>HORIZONTAL RELATIVE DISPLACEMENT (DRIFT) Δ INCHES</th>
<th>MAX. PEAK LOAD-CELL READING kips</th>
<th>Δ/H</th>
<th>ESTIMATED LOAD SURCHARGE AS % OF STANDARD DESIGN LOAD</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TO BEARING CONNECTION</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>TO STUDS IN BEARING CONNECTION</td>
</tr>
<tr>
<td>BF 1</td>
<td>0.122</td>
<td>0.885</td>
<td>0.0010</td>
<td>41</td>
</tr>
<tr>
<td>BF 3</td>
<td>0.266</td>
<td>1.319</td>
<td>0.0022</td>
<td>63</td>
</tr>
<tr>
<td>BF 4</td>
<td>0.437</td>
<td>1.637</td>
<td>0.0036</td>
<td>78</td>
</tr>
<tr>
<td>BF 5</td>
<td>0.623</td>
<td>1.734</td>
<td>0.0052</td>
<td>83</td>
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<td>BF 6</td>
<td>0.967</td>
<td>1.881</td>
<td>0.0081</td>
<td>90</td>
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<tr>
<td>BF 7</td>
<td>1.151</td>
<td>1.808</td>
<td>0.0096</td>
<td>87</td>
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TABLE VIII
TEST II: IN-PLANE CYCLIC TEST OF PRECAST FACADE/CLADDING PANEL AND CONNECTIONS

SUMMARY OF TEST RESULTS - ESTIMATE OF LOAD SURCHARGE TO BEARING CONNECTION

<table>
<thead>
<tr>
<th>THREADED ROD - LENGTH = 8 INCHES. TEST FREQUENCY = 0.1 Hz</th>
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<tr>
<td>RUN</td>
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</tr>
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<td>A 1</td>
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<tr>
<td>A 3</td>
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<tr>
<td>A 4</td>
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<td>A 5</td>
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<table>
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<tbody>
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</tr>
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<tr>
<td>B 7</td>
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<td>B 8</td>
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</table>

TABLE IX
8.3 TEST III  DYNAMIC TESTING OF PRECAST CONCRETE FACADE/CLADDING AND CONNECTIONS IN A MODEL TWO-STORY STEEL MOMENT-RESISTING-FRAME STRUCTURE

8.3.1 TEST OBJECTIVE

The objective of Test III was to experimentally determine the fundamental periods and modal responses of a model two-story steel moment-resisting-frame structure as follows:

- Steel Test-Frame without Cladding Panels
- Steel Test-Frame with Cladding Panels

8.3.2 DESCRIPTION OF TEST SPECIMEN

STEEL TEST STRUCTURE

This test structure is a model two-story one-bay x one-bay steel moment-resisting frame structure with roof/floor system and connections representative of current practice including the base-plate connections at the base. This test structure is a scaled down version of a larger (full-size) steel test structure that was designed sometime back to be tested at an appropriate time at a large earthquake simulator such as the one at U.C. Berkeley. Geometry of the test frame was established by the scaling considerations as well as considerations of laboratory space. The steel test frame was designed to carry a maximum lateral force of 11 kips at roof level in the N-S direction and so as to undergo inter-story drift levels that are significant to investigate the behavior of precast cladding and connections. All beams and columns are W6x9, A-36 steel sections. The test structure was fabricated by a local fabricator and erected in the high-bay laboratory of the School of Architecture. The steel test structure was connected at the bottom to a precast concrete base (bolted to the strong floor slab of the laboratory) using standard base-plate connections that were assumed pinned for analysis and design of the test structure.

Details of the test structure are presented in Figure 1 (Appendix C) and drawing sheets C-4 to C-10 (Appendix C)

Precast Concrete Cladding Panels and Connections

Precast concrete cladding panels were 4-1/2 inches thick, as in Test II and the width and height dimensions of the panels were established so that the mass of the cladding panels expressed as a percentage of the mass of the steel test structure is the same as that in the prototype structure. The cladding panel thickness was kept the same as in Test II so that the cladding connection details will be the same in Test II and III. Details of the precast concrete cladding 2 panel and connections are presented in Figure 5 (Appendix C) and sheets C-6, C-7, C-8 (Appendix C).
The cladding configuration and connection details were developed in consultation with a Precast Manufacturer (89) on the west coast who also fabricated the cladding panels in accordance with current practices of manufacture of architectural precast cladding panels including their connections.

8.3.3 DYNAMIC TEST SET-UP AND PROCEDURE

The test structure was dynamically excited by an APS Electro-Seis shaker positioned on the floor of the test structure. This shaker could be oriented in the Transverse direction (N-S) or the Longitudinal direction (E-W). For study of torsional response characteristics this shaker was positioned 12 inches off-center on the floor of test structure in the Transverse direction (N-S).

Figures 2 and 4 (Appendix C) show photographs of the APS shaker and Test III in progress.

Basically Test III was divided into three separate parts:

- **Test III-A** Steel Test Frame Structure without Precast Cladding Panels.
- **Test III-B** Steel Test Frame Structure with One Precast Cladding Panel attached to east face of the structure.
- **Test III-C** Steel Test Frame Structure with Two Precast Cladding Panels, attached one each to the east and west faces of the structure.

Two types of excitations were used in Test III, as follows:

- **Random Excitation**
  This was provided by the HP Spectrun Analyzer used in this test.

- **Sinusoidal Excitation**
  This was provided by a function generator used in this test.

A schematic block diagram of Dynamic Test Set-up is shown in Fig. 33.

The sequence of the Dynamic Test Runs and Test Control Parameters are presented in Table X.

For each test run the selected excitation was continuously applied and dynamic responses of test structure at roof & floor levels measured by appropriately positioned statham accelerometers.
Modal response of the test structure was obtained by feeding the accelerometer output into the HP Spectrum Analyzer Fig. 3 (Appendix C) which provided a screen display of modal response and then dumping the screen-display down to an x-y plotter.

Figure 1 (Appendix C) shows the overall dynamic test set-up. Figures 2 & 3 (Appendix C) show the test instrumentation and in Test III.

8.3.4 Test Results

A summary of test results obtained for Test III-A (No Cladding Panels) is presented in Table XI.

A summary of test results obtained for Test III-C (Test Structure with Two Cladding Panels in Transverse Direction N-S) is presented in Table XII.
TEST III  DYNAMIC TESTING OF MODEL TEST STRUCTURE WITHOUT & WITH CLADDING PANELS

Figure 33  BLOCK DIAGRAM OF DYNAMIC TEST SET-UP
**TEST III  DYNAMIC TESTING OF MODEL STEEL TEST STRUCTURE**

<table>
<thead>
<tr>
<th>TYPE OF EXCITATION</th>
<th>TEST RUN III-A</th>
<th>TEST RUN III-B</th>
<th>TEST RUN III-C</th>
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</thead>
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<tr>
<td>STEEL TEST FRAME WITHOUT CLADDING PANELS</td>
<td>STEEL TEST FRAME WITH ONE CLADDING PANEL ATTACHED TO EAST FACE OF TEST STRUCTURE</td>
<td>STEEL TEST FRAME WITH TWO PRECAST CLADDING PANELS ATTACHED ONE EACH TO THE EAST AND WEST FACES OF THE TEST STRUCTURE</td>
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</table>

<table>
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<tr>
<th>ORIENTATION OF SHAKER</th>
<th>TRANSVERSE DIRECTION N-S</th>
<th>LONGITUDINAL DIRECTION E-W</th>
<th>OFF-CENTER TRANSVERSE DIRECTION N-S</th>
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<tr>
<td>RANDOM</td>
<td>X</td>
<td>X</td>
<td>X</td>
</tr>
<tr>
<td>SINUSOIDAL</td>
<td>X</td>
<td>X</td>
<td>X</td>
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**TABLE X  DYNAMIC TEST RUNS AND TEST PARAMETERS**
## Table XI

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<th>Mode</th>
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<tr>
<td>Short Direction N-S</td>
<td>7.0 Hz</td>
<td>10.6 Hz</td>
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</tr>
<tr>
<td>Longitudinal Direction E-W</td>
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<td></td>
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<td>Second Translational Mode</td>
<td>19.75 Hz</td>
<td>39.6 Hz</td>
<td></td>
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<tr>
<td>First Torsional Mode</td>
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<td></td>
</tr>
<tr>
<td>Second Torsional Mode</td>
<td></td>
<td>43.0 Hz</td>
<td></td>
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**Summary of Test Results**

Test III-A
TEST III-C  DYNAMIC TEST OF MOMENT-RESISTING STEEL FRAME STRUCTURE WITH TWO PRECAST CLADDING PANELS ATTACHED ONE EACH TO THE EAST AND WEST FACES (SHORT DIRECTION) OF TEST STRUCTURE

<table>
<thead>
<tr>
<th>MODE</th>
<th>NATURAL FREQUENCY Hz</th>
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<tr>
<td></td>
<td>SHORT DIRECTION N-S</td>
<td>LONGITUDINAL DIRECTION E-W</td>
<td>TORSION</td>
</tr>
<tr>
<td>First Translational Mode</td>
<td>5.9 Hz</td>
<td>7.4 Hz</td>
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<td>Second Translational Mode</td>
<td>17.0 Hz</td>
<td>34.5 Hz</td>
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<tr>
<td>First Torsional Mode</td>
<td></td>
<td></td>
<td>9.2 Hz</td>
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<tr>
<td>Second Torsional Mode</td>
<td></td>
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<td>34.8 Hz</td>
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Table XII  SUMMARY OF TEST RESULTS
TEST III-C

59
CHAPTER 9: ANALYTICAL MODELING OF BEHAVIOR OF CLADDING AND CONNECTIONS

• Behavior of Thread-Rod Flexible Connections [TEST I]

Based on an experimentally obtained stress-strain curve for a 5/8-inch diameter threaded-rod, an analytical model for load-deflection prediction of cantilever threaded-rod specimens with support conditions similar to those used in practice, was developed.

Details of the analytical model development process are presented below (p.62-66). Figure 34 shows the assumed stress and strain distribution for the cantilever threaded-rod specimens. A block diagram outlining the steps involved in the analytical prediction model is shown in Figure 35. Details of the derivations required to obtain theoretical moment-curvature relations and load-deflection relations are presented on p.65-66.

A polynomial fit to experimental stress-strain curve for a 5/8-inch diameter threaded steel-rod specimen is presented in Figure 13 (Appendix A).

The moment-curvature curve that was obtained with this analytical model is shown in Figure 14 (Appendix A).

Results in the form of Load-Deflection curves obtained with this analytical prediction model for threaded-rod specimens of 4, 6, 8, 10 and 12 inch lengths are presented in Figures 7-11 (Appendix A).

• In-Plane Behavior of Precast Facades/Claddings and Connection Assemblies [TEST II]

The behavior of full-scale precast facades/claddings and connection assemblies is very complex, especially under cyclic motions. In light of these complexities only practical and simplified analytical evaluation of results of Test II was carried out.

The basic objective of this analytical evaluation was to obtain an overall behavior model, based on cyclic test results of Test II, and compare this model to the conceptual behavior model used in seismic analysis and design of precast cladding and connections.

Based on peak cyclic lateral force and peak cyclic displacements levels reached in each run of Test II, the proposed analytical behavior model was based on the assumption that the peak lateral-force resistance is controlled by the resistance of the top flexible threaded-rod connections. This concept is presented graphically in Figure 36. A seismic design evaluation of the cladding panel and connections was made to determine if any design changes were necessary to account for the lateral-force resistance provided by the threaded-rod flexible connections.

In any case, the fact remains that a great deal of work needs to be done to improve our understanding of the behavior of cladding panels and connections especially under cyclic motions.
The standard design calculation for vertical load \( R \) transferred to bearing connection is given by Eqn. (1).

\[
R = 0.5W + (0.5h)F/b
\]  
(1)

The effect of binding-force \( P \) developed in the threaded-rod lateral connection, on the vertical load \( R' \) transferred to bearing connection is given by Eqn. (2).

\[
R' = 0.5W + (0.5h)F/b + (P)(h)/b
\]  
(2)

The service load-surcharge to bearing angles expressed as a percentage of standard design load is given by Eqn. (3).

\[
= \frac{200}{1 + 0.4h/b} \cdot \frac{P}{W}
\]  
(3)

The service load-surcharge to studs in bearing connection expressed as a percentage of standard design load is given by Fig. (4).

\[
= \frac{200}{1 + 1.2h/b} \cdot \frac{P}{W}
\]  
(4)

where \( W \) = weight of cladding panel  
\( h \) = height of threaded-rod lateral connection from the bearing connection  
\( b \) = horizontal distance between the centerline of the bearing connection  
\( F \) = Standard Seismic Design Load
Modal Response of Two-Story Steel Moment-Resisting Frame Structure With and Without Precast Concrete Cladding Panels (TEST III)

The analytical evaluation of modal response results obtained during Test III is still in progress.

A mode-shape and frequency analysis of the test structure without any cladding panels was carried out using the computer program ETABS, using appropriate modeling to simulate the pinned-base condition assumed. The modal frequencies obtained are presented in Table XIII.
\[ \sigma(y) = \sigma_e \left[ \frac{\varepsilon(y)}{\varepsilon_e} \right]^{n} \quad \varepsilon > \varepsilon_e \quad \text{or} \quad y > y \]

\[ \sigma(y) = \sigma_e \left[ \frac{\varepsilon(y)}{\varepsilon_e} \right] \quad \varepsilon < \varepsilon_e \quad \text{or} \quad y \leq y \]

FIG. 34

ANALYTICAL MODEL FOR LOAD-DEFLECTION PREDICTION

TEST I: TESTS OF THREADED-ROD FLEXIBLE CONNECTIONS
VARIABLES AND PARAMETERS

\[
\begin{align*}
P: & \quad \text{LOAD, LBS.} & \sigma(y): & \quad \text{STRESS AT } y \\
L: & \quad \text{ROD LENGTH - INCHES} & \varepsilon(y): & \quad \text{STRAIN AT } y \\
E: & \quad \text{ROD ELASTICITY} & n: & \quad \text{STRAIN HARDENING EXPONENT} \\
r: & \quad \text{ROD RADIUS, INCHES} & \varepsilon_r: & \quad \text{STRAIN AT } y = r \\
\phi: & \quad \text{CURVATURE} = \varepsilon_r / R = \varepsilon_y / \gamma \\
\delta: & \quad \text{END DEFLECTION OF} \\
\quad \text{ROD ALONE} & \delta_c: & \quad \text{END DEFLECTION DUE} \\
\quad \text{TO ROTATION OF RIGID CONNECTION} & \delta_T: & \quad \delta + \delta_c: \text{TOTAL END DEFLECTION} \\
m(\phi): & \quad \text{MOMENT CORRESPONDING TO CURVATURE } \phi \\
X(P, L; m(\phi)): & \quad \text{LOCATION ALONG ROD CORRESPONDING TO CURVATURE } \phi \\
\sigma_{\text{max}}(P, L): & \quad \text{MAXIMUM STRESS IN ROD FOR GIVEN P, L} \\
CS: & \quad \text{CANTILEVER CONNECTION STIFFNESS, LB-IN/RADIAN} \\
\end{align*}
\]

ANALYTICAL MODEL FOR LOAD-DEFLECTION PREDICTION
TEST I: TESTS OF THREADED-ROD LATERAL CONNECTIONS
FIG 35  BLOCK DIAGRAM
ANALYTICAL MODEL FOR LOAD-DEFLECTION PREDICTION
TEST I: TESTS OF THREADED-ROD LATERAL CONNECTIONS
COMPUTATION OF m(¢) [BLOCK 3]: MOMENT-CURVATURE

(i) For a specific value of Y, the moment m(Y) producing corresponding state of stress/strain is:

\[
    m(Y) = \int_0^r \delta(y) y dA = 2 \int_{-r}^{r} \sigma(y) \sqrt{r^2 - y^2} \, dy
\]

\[
    = 4\sigma_e \left[ \int_0^{y/Y} y \sqrt{r^2 - y^2} \, dy + \int_{y/Y}^{r} y \sqrt{r^2 - y^2} \, dy \right]
\]

(ii) Substitute \( \phi \) for Y where \( Y = \varepsilon_e / \phi = \delta_e / E \phi \)

\[
    m(\phi) = 4\sigma_e \left[ \int_0^{\varepsilon_e / E \phi} (yE/\sigma_e) \sqrt{r^2 - y^2} \, dy + \int_{\varepsilon_e / E \phi}^{r} (yE/\sigma_e) \sqrt{r^2 - y^2} \, dy \right]
\]

(iii) Numerically this is calculated as a sum over I

\[
    m(\phi) = \sum_{I} f(\phi, y_I) \Delta y_I \quad \text{where } \Delta y_I \text{ is incremented from } 0 \text{ to } r
\]
COMPUTATION OF $X(P,L,m(\phi))$ [BLOCK 4]

(i) For cantilever
\[ m = P \times (L-X) \]

(II) Then
\[ X(P,L;m(\phi)) = L - \frac{m(\phi)}{P} \]

COMPUTATION OF $\delta(P,L)$ [BLOCK 5]

(i)
\[ (P,L) = \int_{\phi}^{0} [L-X(\phi)] \times dx(\phi) \]

(ii) Numerically this is calculated using $\phi_j$ as a parameter and summing over $J$:
\[ (P,L) = \sum_{J} \left\{ \phi_j \times [L-X(\phi_j)] + \phi_{j-1} \times [L-X(\phi_{j-1})] \right\} \times [X(\phi_{j-1}) - X(\phi_j)] \]

The moment enters implicitly through $X(P,L;m(\phi))$.

$X$ is computed for sequential $\phi_j$ pairs. The iteration sensitivity is interactively varied to assure adequate precision.

COMPUTATION OF $\sigma_{\text{max}}$ & $\sigma_T$ [BLOCK 5]

(i)
\[ \sigma_{\text{max}}(P,L) = \sigma_e \left( \frac{r}{2Y_{\text{min}}} \right)^n \]

where $2Y_{\text{min}}$ is the depth of elastic zone when $m=m_{\text{max}} = P \times L$

(ii)
\[ \sigma_T(P,L) = \sigma(P,L) + PL^2/CS \]

where $CS$ is the stiffness, Lb.-In./Rad. of the support connection of cantilever threaded rod.
CONCEPTUAL MODEL OF STANDARD DESIGN PROCEDURE FOR VERTICAL LOAD TRANSFER TO BEARING CONNECTIONS

CONCEPTUAL MODEL OF MODIFIED DESIGN PROCEDURE FOR VERTICAL LOAD TRANSFER TO BEARING CONNECTION

Figure 36  Test II
Conceptual Simple Behavior Model for Seismic Analysis and Design of Cladding Panels Connections
ANALYTICAL EVALUATION OF RESULTS OF TEST III-A
MODAL FREQUENCIES OF MOMENT-RESISTING STEEL FRAME STRUCTURE
WITHOUT PRECAST CONCRETE FACADE/CLADDING PANELS

<table>
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<tr>
<th>MODE</th>
<th>NATURAL FREQUENCY Hz</th>
<th>SHORT DIRECTION N-S</th>
<th>LONGITUDINAL DIRECTION E-W</th>
<th>TORSION</th>
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</thead>
<tbody>
<tr>
<td>FIRST TRANSLATIONAL</td>
<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>MODE</td>
<td>4.4 Hz</td>
<td>8.5 Hz</td>
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<td>SECOND TRANSLATIONAL</td>
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<td>SECOND TORSIONAL</td>
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TABLE XIII
CHAPTER 10: DISCUSSION OF RESULTS AND CONCLUSIONS

BEHAVIOR OF LATERAL/THREADED-ROD CONNECTIONS [TEST I]

- A study of the results of Test I specimens shows that load-capacity of threaded-rod cladding connections decreases with increasing length.
- Behavior of threaded-rod specimen in uniaxial tension shows evidence of strain-hardening that must be considered in design and analysis.
- Load-Deflection behavior of cantilever threaded-rod specimens can be predicted using experimentally obtained stress-strain data with reasonably good correlation between experimental and analytical results. Simple elastic beam theory does not appear to be adequate to explain the load-deflection behavior obtained in these static tests.

CYCLIC BEHAVIOR OF PRECAST CONCRETE CLADDING PANELS AND CONNECTION ASSEMBLY [TEST II]

- In-plane resistance of precast concrete cladding panels is controlled by the resistance provided by the threaded-rod lateral connections at top of panels.
- In all cyclic test runs failure occurred in the threaded-rods at the loading-end of top lateral connections.
- The levels of inter-story drift that can be accommodated by the threaded-rod lateral connections can be established from the drifts at failure which varied from 0.0068 H at 0.1 Hz [6-inch threaded-rod length] to 0.0117H at 0.5 Hz [8-inch threaded-rod length].
- Behavior of threaded-rod connections under cyclic displacements shows that further studies are needed to explain the fracturing mechanism of failures observed possibly caused by low-cycle fatigue.
- The lateral-force resistance offered by the threaded-rod lateral connections at top of panels results in a service-load surcharge on the bearing connections at bottom of the panels, which should be taken into account in the seismic design of precast concrete cladding and connection assemblies.

INFLUENCE OF PRECAST CONCRETE CLADDING PANELS ON MODAL RESPONSE OF STEEL FRAME TEST STRUCTURE [TEST III]

- A preliminary study of the results of shaking tests carried out in Test III shows that the addition of precast cladding panels to the test structure reduced the first translation mode frequency from 7 Hz to 5.9 Hz. (approx. 15.71%) and second translational mode frequency from 19.75 Hz to 17 Hz (approx. 13.92%) in the transverse direction, i.e., parallel to the plane of the cladding panels. These preliminary results show that the stiffening effects of precast concrete cladding are significant and must be considered in the seismic design and analysis of buildings.
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<th>1. AUTHOR:</th>
<th>Adham, S.A.</th>
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<td>Out-of-Plane Response of Masonry Walls</td>
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<th>AIA Research Corporation</th>
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<td>TITLE:</td>
<td>Seismic Design for Police and Fire Stations</td>
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<td>KEYWORDS:</td>
<td>Earthquakes, Design, Criteria, Building Components</td>
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<td>Drift and Damage Considerations in Earthquake Resistant Design of Reinforced Concrete Buildings</td>
</tr>
<tr>
<td>SOURCE:</td>
<td>University of Illinois at Urbana-Champaign, 1976</td>
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<tr>
<td>KEYWORDS:</td>
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<th>Allen, Edward</th>
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<td>The Professional Handbook of Building Construction</td>
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<td>SOURCE:</td>
<td>John Wiley &amp; Sons, New York, 1985</td>
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<td>KEYWORDS:</td>
<td>Cladding, Basic Concepts, Design Requirements, Precast Concrete, Curtain Walls, Masonry, Stone, Veneer, Construction, Details, Building Codes, Building Case Studies</td>
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<th>Armhein, James; Kesler, James; Thompson, Leonard; Van Houten, John</th>
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<tbody>
<tr>
<td>TITLE:</td>
<td>Masonry Design Manual</td>
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<tr>
<td>SOURCE:</td>
<td>Masonry Industry Advancement Committee, 1979</td>
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<tr>
<td>KEYWORDS:</td>
<td>Masonry, Veneer, Construction, Details, Practice</td>
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6. AUTHOR: Andersen, Arthur B.
TITLE: Steel Frame, Precast Forms Meet Impossible Time Constraints
SOURCE: Civil Engineering, ASCE, February 1976, pp. 54-56
KEYWORDS: Curtain Walls, Steel Frame, Design, Construction, Practice

7. AUTHOR: Applied Technology Council
SOURCE: Applied Technology Council, Redwood City, California
KEYWORDS: Earthquakes, Regulations, Design Provisions

8. AUTHOR: Armula, Joseph; Brown, Russell
TITLE: Performance Evaluation of Brick Veneer with Steel Stud-Backup
SOURCE: Clemson University, College of Engineering, Department of Civil Engineering, 1982
KEYWORDS: Masonry, Veneer, Steel Studs, Research, Testing

9. AUTHOR: Arnold, Chris; Hopkins, David; Elsesser, Eric
SOURCE: Building System Development, Inc., San Mateo, California
KEYWORDS: Earthquakes, Non-Structural Components, Cladding, Precast Concrete, Glazing, United States, New Zealand, Codes, Standards, Design, Construction Detailing, Practice

10. AUTHOR: Asher, Jefferson W.; Selna, Lawrence G.
TITLE: Anchors for Prefabricated Brick Panels
SOURCE: Proceedings, Third North American Masonry Conference, June
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TITLE: The Interaction Between Exterior Walls and Building Frames in Historic Tall Buildings


KEYWORDS: Facades, Masonry, Building Frame, Interaction, Research

131. AUTHOR: Stockbridge, Jerry G.
TITLE: Woolworth Building Renovation - Precast Concrete Used for Terra Cotta Facade  
SOURCE: PCI Journal, July/August, 1983  
KEYWORDS: Precast Concrete, Terra Cotta, Facade, Renovation

132. AUTHOR: Stockbridge, Jerry G.  
TITLE: Experimental Methods for Evaluation of the Condition of Facades to Resist Seismic Forces  
SOURCE: Eighth World Conference on Earthquake Engineering, San Francisco, California, 1984  
KEYWORDS: Earthquakes, Facades, Conditions Assessment, Testing, Current Practice

133. AUTHOR: Structural Engineers Association of California  
TITLE: Recommended Lateral Force Requirements and Commentary  
SOURCE: Structural Engineers Association of California, San Francisco, California, 1988  
KEYWORDS: Non-Structural Elements, Seismic Design, Connections, Drift

134. AUTHOR: Suter, G.T. and J.S. Hall  
TITLE: How Safe Are Our Cladding Connections?  
SOURCE: Proceedings, First Canadian Masonry Symposium, The University of Calgary, Alberta, Canada, June 7-10, 1976  
KEYWORDS: Cladding, Connections, Masonry, Detailing, Study

135. AUTHOR: Tawresey, John G.  
TITLE: Factors of Safety for Masonry Connections  
KEYWORDS: Masonry, Connections
136. AUTHOR: Teal, E.J.
TITLE: Seismic Drift Control and Building Periods
KEYWORDS: Earthquakes, Drift, Building Period

137. AUTHOR: Teal, E.J.
TITLE: Seismic Drift Control Criteria
KEYWORDS: Earthquakes, Drift, Design, Criteria

138. AUTHOR: Toomath, S. William
TITLE: Architectural Detailing for Earthquake Movement
SOURCE: not listed
KEYWORDS: not listed

139. AUTHOR: Tri-Services, Department of the Army, The Navy and the Air Force
TITLE: Seismic Design for Buildings
SOURCE: Tri-Services, Department of the Army, The Navy and the Air Force
KEYWORDS: Non-Structural Components, Seismic Design, Drift, Exterior Elements and Connections

140. AUTHOR: U.S. Department of Commerce
TITLE: San Fernando, California Earthquake of February 9, 1971, Volumes I, II and III, 1973
KEYWORDS: Earthquakes, Damage, Building Case Studies, Non-Structural Damage

141. AUTHOR: Uchida, N.; Aoyagi, T.; Kawamura, M.; Nakagawa, K.
TITLE: Vibration Test of Steel Frame Having Precast Concrete Panels
KEYWORDS: Precast Concrete, Cladding, Steel Frame, Earthquake, Research Testing

142. AUTHOR: United States Gypsum
TITLE: USG Curtain Wall Systems
SOURCE: United States Gypsum Co., 1979
KEYWORDS: Curtain Walls, Erection, Details

143. AUTHOR: URS/John A. Blume & Associates
TITLE: Seismic Design Guidelines for Essential Buildings
SOURCE: URS/John A. Blume & Associates,
KEYWORDS: Earthquakes, Design Guidelines, Non-Structural Elements, Essential Buildings

144. AUTHOR: Vogel & Meyer Partnership
TITLE: Examples of Calculations and Details
SOURCE: Vogel & Meyer Partnership, Walnut Creek, California, February, 1985
KEYWORDS: Precast Concrete, Cladding, Design, Details, Current Practice

145. AUTHOR: Waddell, Joseph
TITLE: Precast Concrete: Handling and Erection
SOURCE: Iowa State University Press, Ames, Iowa and American Concrete Institute, Detroit, 1974
KEYWORDS: Precast Concrete, Cladding, Erection, Details, Practice

146. AUTHOR: Wallis, Rick W.
TITLE: Designing Glass Panels Against Wind
SOURCE: Civil Engineering, May, 1961
KEYWORDS: Glass, Facades, Wind, Design

147. AUTHOR: Wang, Marcy Li
TITLE: Non-Structural Element Test Phase, U.S. Side Final Report to the National Science Foundation, U.S.-Japan Cooperation Research Project
SOURCE: Center for Environmental Design Research, University of California, Berkeley, California, October, 1986.
KEYWORDS: Earthquakes, Precast Concrete, Cladding, Steel Frames, Connections, Research Testing, Practice

148. AUTHOR: Wang, Marcy Li
TITLE: Cladding Performance on a Large Scale Test Frame
KEYWORDS: Earthquakes, Precast Concrete, Cladding, Steel Frames, Connections, Research Testing, Practice

149. AUTHOR: Weidlinger, Paul
TITLE: Shear Field Panel Bracing
KEYWORDS: Concrete (precast), Frames, Panels, Shear, Steel Structures

150. AUTHOR: Whitlock, A. Rhett; Brown, Russell H.
TITLE: Cyclic and Monotonic Strength of Anchor Bolts in Masonry
SOURCE: Proceeding, Third North American Masonry Conference, June 1985, University of Texas, Arlington, Texas
KEYWORDS: Anchorage, Masonry, Testing

151. AUTHOR: Wilden, H. & Associates
TITLE: Examples of Precast Panel Details
SOURCE: H. Wilden & Associates, 1984
KEYWORDS: Precast Concrete, Cladding, Design, Details, Current Practice

152. AUTHOR: Yura, Joseph A.
TITLE: Differential Movement Effects in Steel Shelf Angles
KEYWORDS: Connections, Masonry, Cladding, Structural Analysis
APPENDIX A TEST I

DRAWINGS OF TEST SET-UP AND TEST SPECIMEN
PHOTOGRAPHS
GRAPHS OF TEST RESULTS
Fig. 1 NSF SPONSORED RESEARCH PROJECT: SEISMIC TESTING OF HEAVY PRECAST FACADES / CLADDING AND CONNECTIONS IN STEEL BUILDINGS WITH MOMENT RESISTING FRAMES

TEST 1 DUCTILE CONNECTION TEST SET-UP

SEISMIC TEST FACILITY
ARCHITECTURAL ENGR. DEPT.
CAL POLY STATE UNIVERSITY
SAN LUIS OBISPO, CA. 93401

DRAWN: GIL DATE: 08/77
CHCK: DATE:
Figure 2 Test I - Connection Assembly

Figure 3 Test I - Connection Assembly Showing Placement of Threaded-Rod Specimen and The Loading-Tee
Figure 4 Test I: Test Set-up, Threaded-Rod Specimen Length=4 Inches After Load-Test

Figure 5 Test I: Test Set-up, Threaded-Rod Specimen Length=12 Inches Before Load-Test
UNIAXIAL TENSILE TEST OF 5/8 INCH DIA. THREADED ROD

MODULUS OF ELASTICITY $E = 23.35$ KSI

FIG. 6 GRAPH OF UNIAXIAL TENSILE STRESS VS. STRAIN
5/8 INCH DIAMETER THREADED-ROD SPECIMEN
FIG. 7 LOAD VS. DISPLACEMENT CURVES
THREADING-ROD LENGTH = 4 INCHES
FIG. 9 LOAD VS. DISPLACEMENT CURVES
THREADED-ROD LENGTH = 8 INCHES
Fig. 10  LOAD VS. DISPLACEMENT CURVES
THREADED-ROD LENGTH = 10 INCHES

TEST CST-L10A  TEST CST-L10  ANALYTICAL MODEL

LOAD DEFLECTION, INCHES  LOAD, LBS.
FIG. 11 LOAD VS. DISPLACEMENT CURVES
THREADED-ROD LENGTH = 12 INCHES

TEST CST-L12A
TEST CST-L12
ANALYTICAL MODEL

ROD DEFLECTION, INCHES

LOAD, LBS.
FIG. 12 ESTIMATED BENDING STIFFNESS OF THREADED-ROD SPECIMENS AT VARIOUS LOAD LEVELS
UNIAXIAL TENSILE TEST OF 5/8 INCH DIA. THREADED ROD

POLYNOMIAL FIT TO EXPERIMENTAL DATA

\[
\sigma = 61 (\epsilon / 0.00263) \quad \text{FOR} \quad \epsilon < 0.00263
\]

\[
\sigma = 61 (\epsilon / 0.00263)^{0.07} \quad \text{FOR} \quad \epsilon > 0.00263
\]

FIG. 13 STRESS-STRAIN CURVE FOR 5/8 INCH THREADED-ROD USED FOR ANALYTICAL MODELING
TEST I: STATIC BENDING TESTS OF THREADED-ROD LATERAL CONNECTIONS
MOMENT-CURVATURE RELATIONSHIP
5/8 INCH DIA. THREADED ROD

FIG. 14  PREDICTED MOMENT-CURVATURE CURVE - 5/8 INCH DIAMETER THREADED-ROD
APPENDIX B  TEST II

DRAWINGS OF TEST SET-UP AND TEST SPECIMEN

PHOTOGRAPHS

TIME HISTORY PLOTS

GRAPHS OF TEST RESULTS
**ELEVATION-TEST SET-UP**

**Fig. 1**

**NSF SPONSORED RESEARCH PROJECT:**
SEISMIC TESTING OF HEAVY PRECAST FACADES/CLADDING AND CONNECTIONS IN STEEL BUILDINGS WITH MOMENT RESISTING FRAMES

**TEST II**
IN-PLANE CYCLIC TESTING OF PRECAST FACADE/CLADDING PANEL & CONNECTIONS

**SEISMIC TEST FACILITY**
ARCHITECTURAL ENGR. DEPT.
CAL POLY STATE UNIVERSITY
SAN LUIS OBISPO, CA. 93401

**DRAWN:**
**DATE:**
**CHCK:**
**DATE:**
NSF SPONSORED RESEARCH PROJECT:
SEISMIC TESTING OF HEAVY PRECAST FACADES/
CLADDING AND CONNECTIONS IN STEEL BUILDINGS
WITH MOMENT RESISTING FRAMES

TEST II
IN-PLANE CYCLIC TESTING OF PRECAST
FACADE/CLADDING PANEL + CONNECTIONS

SEISMIC TEST FACILITY
ARCHITECTURAL ENGR. DEPT.
CAL POLY STATE UNIVERSITY
SAN LUIS OBISPO, CA. 93401

DRWN: CIL
DATE: 7/81
CHECK:
DATE:
**Fig. 3**

**NSF SPONSORED RESEARCH PROJECT:**
Seismic Testing of Heavy Precast Facades/Cladding and Connections in Steel Buildings with Moment Resisting Frames

**TEST II**
In-plane Cyclic Testing of Precast Facade/Cladding Panel + Connections

**SEISMIC TEST FACILITY**
Architectural Engr. Dept.
CAL POLY STATE UNIVERSITY
SAN LUIS OBISPO, CA. 93401

**DRWN:** CIL
**DATE:** 1/67

**CHECK:**
**DATE:**
Figure 4  Overall View of Full-Size Precast Concrete Cladding Panel Test Specimen and Cyclic Test Set-up
Figure 5  Rigid Bearing Connection
Close-up View

Figure 6  Test Instrumentation
Figure 7  Typical Cyclic Behaviour of Threaded-Rod Lateral Connection at Top Specimen FRCRT-L8; Run No. AF-6 Amplitude=±1.5 Inches; Frequency=0.1 Hz

Figure 8  Overall View of Upper Portion of Cladding Panel Typical Failure of Threaded-Rod Connections at Top Cyclic Test Specimen FPCRT-L8; Run No. AF-8 Amplitude=±2 Inches; Frequency=0.1 Hz
Figure 9  Overall View of Typical Failure of Threaded-Rod Lateral Connections at Top Cyclic Test Specimen FPCRT-L6; Run No. A6
Figure 10  Typical Failure of Threaded-Rod Connection at Top Cyclic Cladding Test Specimen FPCRT-L8; Run No. AF-8 Amplitude=±2 Inches; Frequency=0.1 Hz
TEST II: IN-PLANE CYCLIC TEST OF PRECAST FACADE/CLADDING PANEL & CONNECTIONS

TEST: FPCRT-L6  RUN: AF6  RANGE  YMIN: -1612.1150
CHANNEL: 4  FORCE-POUNDS  YMAX: 1758.6710
NA  GRID LOAD CELL

Figure 11  Time-History Plot of Load
Cladding Specimen FPCRT-L6; Run AF-6
Figure 12 Time-History Plot of Top Horizontal Displacement
Cladding Specimen FPCRT-L6; Run AF-6
Figure 13 Time-History Plot of Strain
Bottom Left Vertical Strain Gage
Cladding Specimen FPCRT-L6; Run AF-6
Figure 14 Time-History Plot of Strain
Bottom Left Horizontal Strain Gage
Cladding Specimen FPCRT-L6; Run AF-6
Figure 15  Time-History Plot of Strain
Bottom Right Vertical Strain Gage
Cladding Specimen FPCRT-L6; Run AF-6
TEST II: IN-PLANE CYCLIC TEST OF PRECAST FACADE/CLADDING PANEL & CONNECTIONS
TEST: FPCRT-L8  RUN: B4  RANGE YMIN: -1050.3180
CHANNEL: 4  FORCE-POUNDS  YMAX: 952.6136
NA  GRID LOAD CELL

Figure 16  Time-History Plot of Load
Cladding Specimen FPCRT-L8; Run B4
TEST II: IN-PLANE CYCLIC TEST OF PRECAST FACADE/CLADDING PANEL & CONNECTIONS
TEST: FPCRT-L8 RUN: B4 RANGE YMIN: -.5056
CHANNEL: 8 INCHES YMAX: .4470
LVDT 2 PANEL TO GRID LVDT TOP RIGHT

Figure 17 Time-History Plot of Top Horizontal Displacement
Cladding Specimen FPCRT-L8; Run B4
Figure 18 Time-History Plot of Strain
Bottom Left Vertical Strain Gage
Cladding Specimen FPCRT-L8; Run B4
TEST II: IN-PLANE CYCLIC TEST OF PRECAST FACADE/CLADDING PANEL & CONNECTIONS

TEST: FPCRT-L8    RUN: B4    RANGE: YMIN: -12.2130
CHANNEL: 3        MICROSTRAIN: YMAX: 19.5400
HORR:             BOTTOM LEFT HORIZONTAL STRAIN GAGE

Figure 19  Time-History Plot of Strain
Bottom Left Horizontal Strain Gage
Cladding Specimen FPCRT-L8; Run B4
Figure 20  Time-History Plot of Strain
Bottom Right Vertical Strain Gage
Cladding Specimen FPCRT-L8; Run B4
Figure 21
FULL PANEL CYCLIC RACKING TEST
SPECIMEN FPCRT-L6
RUN NO. AF-6
FREQUENCY = 0.1 Hz
PEAK COMMAND DISPLACEMENT = ±1½"
TEST II: IN-PLANE CYCLIC TESTING OF PRECAST FACADE/CLADDING PANEL & CONNECTIONS

Fig. 22 PEAK LOAD VS. PEAK DISPLACEMENT CURVES
FIG. 23

PEAK LATERAL FORCE RESISTANCE OF THREADED-ROD LATERAL CONNECTIONS VS. DRIFT

SEISMIC TESTING FACILITY, ARCHITECTURAL ENGINEERING DEPARTMENT
SAN LUIS OBISPO, CA 93407

SEISMICALLY SPONSORED RESEARCH PROJECT
SEISMIC TESTING FACADE/CLADDING PANELS & CONNECTIONS

TEST II IN-PLANE CYCLIC TESTING OF PRECAST CONCRETE FACADE/CLADDING PANEL & CONNECTIONS

PEAK LATERAL FORCE RESISTANCE (P/P,0) VS. DRIFT (ΔM/ΔM,0)

PLotted for varying frequencies:
- Frequency = 0.05 Hz
- Frequency = 0.1 Hz
- Frequency = 0.5 Hz

Drift/Story Height (ΔM) vs. PEAK LATERAL FORCE RESISTANCE (P/P,0)
FIG. 24

PEAK LOAD SURCHARGE TO BEARING
CONNECTION ANGLE VS. DRIFT
TEST FREQUENCY = 0.1 Hz

NSF SPONSORED RESEARCH PROJECT
SEISMIC TESTING OF PRECAST CONCRETE
FAÇADES/CLADDING & CONNECTIONS

SEISMIC TEST FACILITY
ARCHITECTURAL ENGINEERING DEPARTMENT
CAL POLY STATE UNIVERSITY
SAN LUIS OBISPO, CA 93407
DRAWN____ DATE____ CHECKED____ DATE____

B-22
FIG. 25

PEAK LOAD SURCHARGE TO BEARING CONNECTION ANGLE VS. DRIFT

TEST FREQUENCY = 0.5 Hz

SAN LUIS OBISPO, CA 93407

TEST II IN-PLANE CYCLIC TESTING OF PRECAST CONCRETE FACADE/CLADDING PANEL & CONNECTIONS

% PEAK LOAD SURCHARGE TO BEARING CONNECTION ANGLE

0 0.001 0.002 0.003 0.004 0.005 0.006 0.007 0.008 0.009 0.01

Δ/H DRIFT/STORY-HEIGHT

FREQUENCY = 0.5 Hz

FREQUENCY = 0.5 Hz
FIG. 26

PEAK LOAD SURCHARGE TO STUDS IN BEARING CONNECTION VS. DRIFT TEST FREQUENCY = 0.1 Hz

ARCHITECTURAL ENGINEERING DEPARTMENT
SAN LUIS OBISPO, CA 93407

DRAWN DATE CHECKED DATE

B-24

TEST II IN-PLANE CYCLIC TESTING OF PRECAST CONCRETE FACADE/CLADDING PANEL & CONNECTIONS

Δ/H DRIFT/STORY-HEIGHT
FIG. 27

PEAK LOAD SURCHARGE TO STUDS IN BEARING CONNECTION VS. DRIFT
TEST FREQUENCY = 0.5 Hz

NSF SPONSORED RESEARCH PROJECT
SEISMIC TESTING OF PRECAST CONCRETE FACADES/CLADDING & CONNECTIONS

SEISMIC TEST FACILITY
ARCHITECTURAL ENGINEERING DEPARTMENT
CAL POLY STATE UNIVERSITY
SAN LUIS OBISPO, CA 93407
DRAWN DATE CHECKED DATE
CALIFORNIA POLYTECHNIC STATE UNIVERSITY, SAN LUIS OBISPO, CA 93407
ARCHITECTURAL ENGINEERING DEPARTMENT - HIGH BAY LAB

TEST II: IN-PLANE CYCLIC TEST OF PRECAST FACADE/CLADDING PANEL AND CONNECTIONS

PRINCIPAL INVESTIGATOR: SAT RIHAL, ARCHITECTURAL ENGINEERING DEPARTMENT
FACULTY ASSOCIATE: GARY GRANEMAN, ET/EL DEPARTMENT (TESTING AND INSTRUMENTATION)
STUDENT RESEARCH ASSISTANTS: KURT J. CLANDENING AND DWAYNE P. SLAVIN, ARCE SENIORS
TECHNICIAN: BOB MYERS

TEST SCHEDULE: SPECIMEN NO: FPCRT-16 DATE: 8/27/87 TIME: 1:30 PM LENGTH OF THREADED ROD: 6" PANEL THICKNESS: 4-1/2" PANEL SIZE: 8'w x 10'h
GENERAL DESCRIPTION: In-plane Cyclic Test of Full Size Precast Concrete Cladding Panel with Threaded-Rod Lateral Connections @ Top & Bearing Connections @ Bottom

<table>
<thead>
<tr>
<th>RUN NO.</th>
<th>CYCLIC</th>
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**COMMAND PEAK DISPLACEMENT OF CYCLES (INCHES)**

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<th>CYCLE</th>
<th>COMMAND PEAK DISPLACEMENT</th>
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<tr>
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<td>±3/8&quot;</td>
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<tr>
<td>3</td>
<td>±1/2&quot;</td>
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<tr>
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<tr>
<td>6</td>
<td>±1-1/2&quot;</td>
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<tr>
<td>7</td>
<td>±1-3/4&quot;</td>
</tr>
<tr>
<td>8</td>
<td>±2-1/2&quot;</td>
</tr>
</tbody>
</table>

**X-Y RECORDER**
- X: NO DATA
- Y: FAILURE

**MAG-TAPE READING**
- INITIAL
- FINAL

**TRANSDUCER**
- VISI + VESICTION
- PC + PERSONAL COMPUTER
- CH # CH #

<table>
<thead>
<tr>
<th>CHANNEL</th>
<th>ACCELEROMETER GRID - AXIAL</th>
<th>PANEL BOTTOM LEFT CONNECTION VERTICAL STRAIN GAGE</th>
<th>PANEL BOTTOM LEFT CONNECTION HORIZONTAL STRAIN GAGE</th>
<th>LOAD CELL</th>
<th>PANEL BOTTOM RIGHT CONNECTION VERTICAL STRAIN GAGE</th>
<th>PANEL BOTTOM RIGHT CONNECTION HORIZONTAL STRAIN GAGE</th>
<th>LVDT - GRID AXIAL</th>
<th>PANEL TOP LEFT CONNECTION VERTICAL POTENTIOMETER (P)</th>
<th>PANEL TOP RIGHT CONNECTION VERTICAL POTENTIOMETER (G)</th>
<th>LVDT - PANEL BOTTOM</th>
</tr>
</thead>
<tbody>
<tr>
<td>CH</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
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<td>±135</td>
<td>±1974</td>
<td>±169</td>
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<td>VISI/PC/CH</td>
<td>VISI/PC/CH</td>
<td>VISI/PC/CH</td>
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<td>VISI/PC/CH</td>
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<td>VISI/PC/CH</td>
<td>VISI/PC/CH</td>
<td>VISI/PC/CH</td>
</tr>
</tbody>
</table>

Table I
**CALIFORNIA POLYTECHNIC STATE UNIVERSITY, SAN LUIS OBISPO, CA 93407**
ARCHITECTURAL ENGINEERING DEPARTMENT - HIGH BAY LAB

**TEST I: IN-PLANE CYCLIC TEST OF PRECAST FACADE/CLADDING PANEL AND CONNECTIONS**

**PRINCIPAL INVESTIGATOR:** SAT RIHAL, ARCHITECTURAL ENGINEERING DEPARTMENT  
**FACULTY ASSOCIATE:** GARY GRANNEMAN, ETIEL DEPARTMENT (TESTING AND INSTRUMENTATION)  
**STUDENT RESEARCH ASSISTANTS:** KURT J. CLANDENING AND DWAYNE P. SLAVIN, ARCE SENIORS  
**TECHNICIAN:** BOB MYERS

**TEST SCHEDULE:** SPECIMEN NO: FPCRT-L6  
**DATE:** 8/27/87  
**TIME:** 2:45 PM  
**LENGTH OF THREADED ROD:** 6"  
**PANEL THICKNESS:** 4-1/2"  
**PANEL SIZE:** 8'w x 10'h

**GENERAL DESCRIPTION:** In-Plane Cyclic Test of Full Size Precast Concrete Cladding Panel with Threaded Rod Lateral Connections @ Top & Bearing Connections @ Bottom

<table>
<thead>
<tr>
<th>ROW NO.</th>
<th>FREQUENCY</th>
<th>COMMAND PEAK DISPLACEMENT OF CYCLES (INCHES)</th>
<th>REMARK</th>
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</thead>
<tbody>
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<td></td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>±3/8&quot;</td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>2</td>
<td>±1/2&quot;</td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>3</td>
<td>±3/4&quot;</td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>4</td>
<td>±1&quot;</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>5</td>
<td>±1-1/2&quot;</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>±2&quot;</td>
<td></td>
</tr>
<tr>
<td>8</td>
<td>7</td>
<td>±2-1/2&quot;</td>
<td>FAILURE</td>
</tr>
</tbody>
</table>

**LOAD CELL**

| X-Y RECORDER LOAD CELL | X mV/in | 100 | 100 | 100 | 100 |
| LVDT 1 | Y mV/in | 100 | 100 | 250 | 250 | 250/1000 | 1000 | 1000 |

**MAG-TAPE READING**

**INITIAL**  
| PANEL BOTTOM LEFT CONNECTION VERTICAL STRAIN GAGE | 114 | ±102 | ±176 | ±214 | ±232 | ±254 | ±200 | ±120 |
| PANEL BOTTOM LEFT CONNECTION VERTICAL STRAIN GAGE | 115 | ±27 | ±47 | ±59 | ±60 | ±66 | ±52 | ±60 |
| LOAD CELL | LB. | 116 | ±1759 | ±1270 | ±1514 | ±1624 | ±1759 | ±1348 | ±880 |
| PANEL BOTTOM RIGHT CONNECTION VERTICAL STRAIN GAGE | 117 | ±133 | ±234 | ±278 | ±300 | ±311 | ±240 |
| PANEL BOTTOM RIGHT CONNECTION HORIZONTAL STRAIN GAGE | 118 | drift | drift | drift | drift | drift | drift | drift |
| LVDT - GRID AXIAL inches | 119 | ±0.156 | ±0.331 | ±0.532 | ±0.716 | ±1.06 | ±1.19 | ±1.36 |
| LVDT - PANEL TOP inches | 120 | ±0.109 | ±0.248 | ±0.416 | ±0.591 | ±0.912 | ±1.058 | ±1.41/1.22 |
| PANEL TOP LEFT CONNECTION VERTICAL POTENTIOMETER (P) | inches | ±0.004 | ±0.011 | ±0.017 | ±0.027 | ±0.034 | ±0.044 | ±0.048/0.054 |
| PANEL TOP RIGHT CONNECTION VERTICAL POTENTIOMETER (G) | inches | ±0.004 | ±0.011 | ±0.017 | - | ±0.034 | ±0.044 | ±0.048/0.054 |
| LVDT - PANEL BOTTOM inches | 121 | very small | very small | very small | very small | very small | very small | very small |
CALIFORNIA POLYTECHNIC STATE UNIVERSITY, SAN LUIS OBISPO, CA 93407
ARCHITECTURAL ENGINEERING DEPARTMENT - HIGH BAY Lab

TEST II: IN-PLANE CYCLIC TEST OF PRECAST FACADE/CLADDING PANEL AND CONNECTIONS

PRINCIPAL INVESTIGATOR: SAT RIHAL, ARCHITECTURAL ENGINEERING DEPARTMENT
FACULTY ASSOCIATE: GARY GRANNEMAN, ET/EL DEPARTMENT (TESTING AND INSTRUMENTATION)
STUDENT RESEARCH ASSISTANTS: KURT J. CLANDENING AND DWAYNE P. SLAVIN, ARCE SENIORS
TECHNICIAN: BOB MYERS

TEST SCHEDULE: SPECIMEN NO: FPCLT-18  DATE: 8/25/87  TIME:
LENGTH OF THREADED ROD: 8"  PANEL THICKNESS: 4-1/2"  PANEL SIZE: 8'w x 10'h
GENERAL DESCRIPTION: In-Plane Cyclic Test of Full Size Precast Concrete Cladding Panel with Threaded-Rod Lateral Connections @ Top & Bearing Connections @ Bottom

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<th>INPUT FREQUENCY</th>
<th>CYCLES</th>
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<th>MAG-TAPE READING</th>
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<th>PC = PERSONAL COMPUTER</th>
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<td>g's</td>
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<td>LBS</td>
<td>2 2</td>
<td>1110</td>
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<td>PANEL BOTTOM LEFT CONNECTION HORIZONTAL STRAIN GAGE</td>
<td>LBS</td>
<td>3 3</td>
<td>±22</td>
<td>±34</td>
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<tr>
<td>LOAD CELL</td>
<td>LBS</td>
<td>4 4</td>
<td>±733</td>
<td>±1103</td>
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<td>PANEL BOTTOM RIGHT CONNECTION VERTICAL STRAIN GAGE</td>
<td>LBS</td>
<td>5 5</td>
<td>±125</td>
<td>±188</td>
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<td>LBS</td>
<td>6 6</td>
<td>drift</td>
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<tr>
<td>LVDT - GRID AXIAL</td>
<td>inches</td>
<td>7 7</td>
<td>±0.233</td>
<td>±0.463</td>
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<tr>
<td>LVDT - PANEL TOP</td>
<td>inches</td>
<td>8 8</td>
<td>±0.181</td>
<td>±0.380</td>
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<tr>
<td>PANEL TOP LEFT CONNECTION VERTICAL POTENTIOMETER (P)</td>
<td>inches</td>
<td>11</td>
<td>±0.009</td>
<td>±0.016</td>
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<tr>
<td>PANEL TOP RIGHT CONNECTION VERTICAL POTENTIOMETER (P)</td>
<td>inches</td>
<td>10</td>
<td>±0.009</td>
<td>±0.016</td>
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<tr>
<td>LVDT - PANEL BOTTOM</td>
<td>inches</td>
<td>9 very small</td>
<td>-</td>
<td>very small</td>
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<td>LOAD CELL</td>
<td>LBS</td>
<td>12</td>
<td>±1203</td>
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Table III
### Test II: In-Plane Cyclic Test of Precast Façade/Cladding Panel and Connections

**Principal Investigator:** Sat Rihal, Architectural Engineering Department  
**Student Research Assistants:** Kurt J. Clandening and Dwayne P. Slavin, ARCE Seniors  
**Technician:** Bob Myers

**Test Schedule:**  
Specimen No: FPCR-L8  
Date: 8/26/87  
Time: 5:25 PM  
Length of Threaded Rod: ~8"  
Panel Size: 8'w x 10'h  
Panel Thickness: 4-1/2"  
Panel Thickness: 8-1/4"

**General Description:** In-plane Cyclic Test of Full Size Precast Concrete Cladding Panel with Threaded-Rod Lateral Connections @ Top & Bearing Connections @ Bottom

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<th>REMARK</th>
<th>DATA FILE NAMES</th>
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<td>B</td>
<td>8</td>
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#### Command Peak Displacement of Cycles (Inches)

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<td>±1-3/4&quot;</td>
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<th>PEAK DISPLACEMENT OF CYCLES (INCHES)</th>
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<tr>
<td>X mV/in</td>
<td>100 100 100 100 100 100 100 100</td>
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<tr>
<td>y mV/in</td>
<td>100 100 200 500 500 500 500 500</td>
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<td>FINAL</td>
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<td>1404 1436 1451 1466 1483 1500</td>
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Table IV
APPENDIX C TEST III

PHOTOGRAPHS

DRAWINGS OF TEST STRUCTURE AND CLADDING PANELS AND CONNECTION DETAILS

TYPICAL OUTPUT FROM ANALYZER
Figure 1  Photograph - Two-Story Moment-Resisting Rigid-Frame Test Structure for Dynamic Testing of Cladding and Connections

Figure 2  Photograph - APS Electro-Seis Shaker Positioned in the Floor of Test Structure in the N-S Direction
Figure 3 Photograph - Test Instrumentation
HP3582A Spectrum Analyzer

Figure 4 Photograph - Dynamic Test of Test-Structure Without Cladding Panels
APS Electro-Seis Shaker Positioned on Floor in the N-S Direction
Figure 5  Photograph - 4\(\frac{1}{2}\)-inch Thick Precast Cladding Panel Before Attachment to the Test III Steel Test Frame Structure
FIRST AND SECOND FLOOR FRAMING PLAN

SCALE ½"=1'-0"

LEGEND

BM-BM. CONNECTION DETAIL

SCALE : 5"=1'-0"

NSF SPONSORED RESEARCH PROJECT:
SEISMIC TESTING OF HEAVY PRECAST FACADES/CLADDINGS AND CONNECTIONS IN STEEL BUILDINGS WITH MOMENT RESISTING FRAMES

TEST III
DYNAMIC TESTING OF PRECAST FACADE/CLADDING PANEL AND CONNECTIONS ON A MODEL TWO-STORY STEEL MOMENT-RESISTING FRAME STRUCTURE

SEISMIC TEST FACILITY
ARCHITECTURAL ENGR. DEPT.
CAL POLY STATE UNIVERSITY
SAN LUIS OBISPO, CA 93407

DRWN: DATE:
NORTH/SOUTH ELEV.

EAST/WEST ELEV.

10⅞ x 5⅞ x ¾ BASE PLATE (Typ.)
PRECAST CONC. BASE
1½ ø HOLD DOWN BOLTS

LAB FLOOR
BEARING CONNECTION - SECTION

NSF SPONSORED RESEARCH PROJECT:
SEISMIC TESTING OF HEAVY PRECAST FACADES/CLADDING AND CONNECTIONS IN STEEL BUILDINGS
WITH MOMENT RESISTING FRAMES

BEARING CONNECTION - PLAN
FOUNDATION PLAN

SHEET 6

NSF SPONSORED RESEARCH PROJECT:
SEISMIC TESTING OF HEAVY PRECAST FACADES/CLADDINGS AND CONNECTIONS IN STEEL BUILDINGS WITH MOMENT RESISTING FRAMES

TEST III
DYNAMIC TESTING OF PRECAST FACADE/CLADDING PANEL AND CONNECTIONS ON A MODEL TWO-STORY STEEL MOMENT-RESISTING FRAME STRUCTURE

SEISMIC TEST FACILITY
ARCHITECTURAL ENGR. DEPT.

DRWN: CAL POLY STATE UNIVERSITY
DATE: SAN LUIS OBISPO, CA 93407
Figure 6 TEST III
Typical Printout of Spectrum Analyzer
Display
Test Run III-C
Short Direction
Random Excitation