NOTICE

Seismic Resistance of Slab-Column Connections in Existing Non-Ductile Flat-Plate Buildings

by

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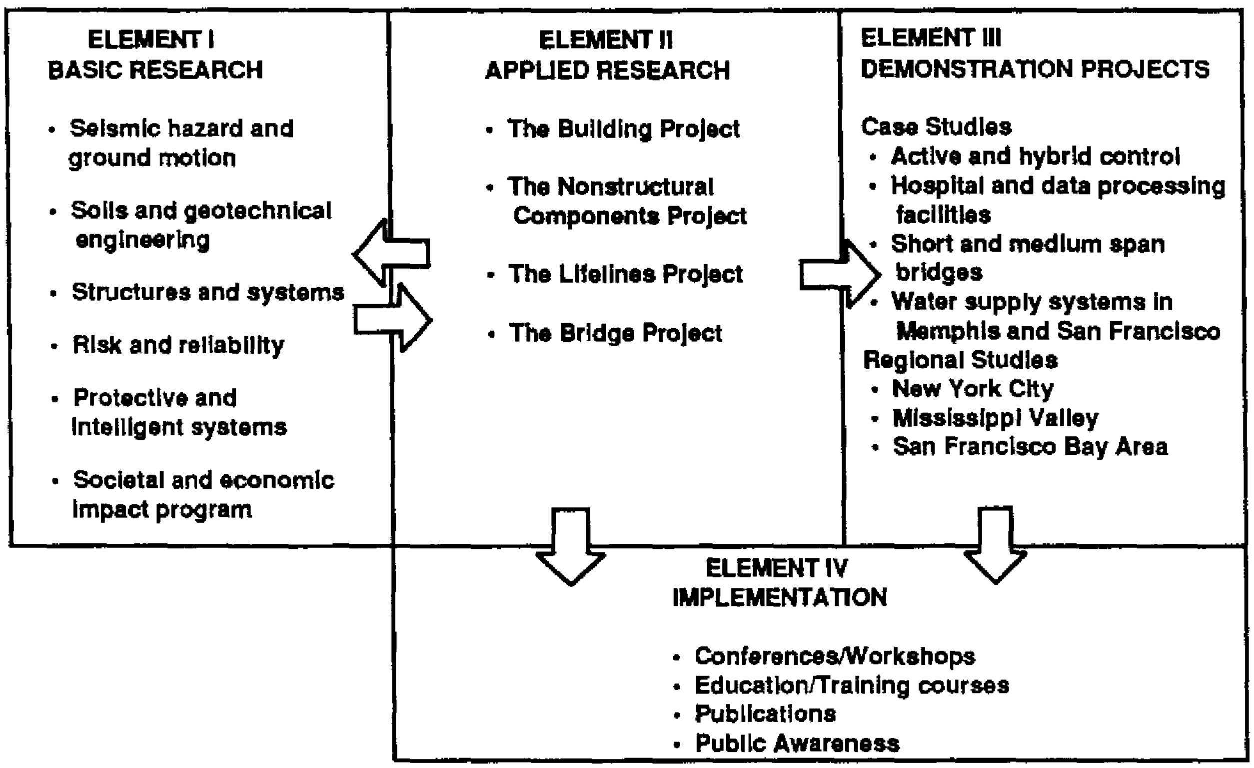
PREFACE

The National Center for Earthquake Engineering Research (NCEER) was established to expand and disseminate knowledge about earthquakes, improve earthquake-resistant design, and implement seismic hazard mitigation procedures to minimize loss of lives and property. The emphasis is on structures in the eastern and central United States and lifelines throughout the country that are found in zones of low, moderate, and high seismicity throughout the United States.

NCEER's research and implementation plan in years six through ten (1991-1996) comprises four interlocked elements, as shown in the figure below. Element I, Basic Research, is carried out to support projects in the Applied Research area. Element II, Applied Research, is the major focus of work for years six through ten. Element III, Demonstration Projects, have been planned to support Applied Research projects, and will be either case studies or regional studies. Element IV, Implementation, will result from activity in the four Applied Research projects, and from Demonstration Projects.



Research in the **Building Project** focuses on the evaluation and retrofit of buildings in regions of moderate seismicity. Emphasis is on lightly reinforced concrete buildings, steel semi-rigid frames, and masonry walls or infills. The research involves small- and medium-scale shake table tests and full-scale component tests at several institutions. In a parallel effort, analytical models and computer programs are being developed to aid in the prediction of the response of these buildings to various types of ground motion.

Two of the short-term products of the **Building Project** will be a monograph on the evaluation of lightly reinforced concrete buildings and a state-of-the-art report on unreinforced masonry.

The **structures and systems program** constitutes one of the important areas of research in the **Building Project**. Current tasks include the following:

1. Continued testing of lightly reinforced concrete external joints.
2. Continued development of analytical tools, such as system identification, idealization, and computer programs.
3. Perform parametric studies of building response.
4. Retrofit of lightly reinforced concrete frames, flat plates and unreinforced masonry.
5. Enhancement of the IDARC (inelastic damage analysis of reinforced concrete) computer program.
6. Research infilled frames, including the development of an experimental program, development of analytical models and response simulation.
7. Investigate the torsional response of symmetrical buildings.

As part of the Lightly Reinforced Concrete (LRC) area, this project has addressed the performance of existing flat-plate buildings, which are very common in most parts of the country. This report addresses the first step, the evaluation of existing flat-plate buildings; a follow-up report will be concerned with their retrofit. Companion reports summarize work on LRC frames. The experiments described in this report have shown that the lateral resistance of flat plates can be governed by the punching failure of interior connections, the flexural yielding of the slab, and the cracking strength of the slab under positive moment. This investigation showed the importance of the gravity load. The report provides guidelines about the expected capacity and maximum drift.

ABSTRACT

An experimental investigation was conducted to evaluate the seismic resistance of slab-column connections in existing non-ductile flat-plate buildings. The test subassemblies were designed and detailed in accordance with the building codes of the late forties and fifties. Each subassembly consisted of two exterior and one interior connection and was subjected to several cycles of increasing lateral displacements. The test variables included the slab reinforcing detail, intensity of the gravity load applied to the slab, and the presence of a spandrel beam.

Rapid stiffness degradation, significant reduction in deformation capacity under increased gravity load, and limited moment-transfer capacity of connections were observed to be the main response characteristics of the non-ductile slab-column connections subjected to earthquake-type loading. The connections were able to sustain full design dead and live loads through at least 2% drift. However, 70% of the initial lateral stiffness of the connections was lost by this drift level. Under normal service loads, the connections were able to maintain at least 80% of their strength through approximately 4% lateral drift and the transfer of unbalanced moment occurred mainly on the column face with slab under negative bending.

The mode of failure and deformation capacity of the connections was observed to depend greatly on intensity of the gravity load on the slab. At lower gravity loads, the response of connections was dominated by flexural yielding of the slab. As the gravity load was increased, the interior connections punched at significantly smaller lateral drift. Provided the gravity shear is kept within a certain maximum limit, the test results suggest that the rapid degradation of the lateral stiffness and the lack of protection against progressive collapse resulting from punching of connections may be the two most critical factors affecting the response of non-ductile flat-plate buildings during moderate earthquakes.

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TABLE OF CONTENTS

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF ILLUSTRATIONS (cont'd)

LIST OF TABLES

SECTION 1

INTRODUCTION

The Eastern United States has a large inventory of older flat-slab buildings which were designed and built to resist gravity loads only. Recent concerns over the probability of a moderate size earthquake in this region necessitate the evaluation of the damage potential of these buildings. Buildings of this type are typically five to fifteen stories high and are mainly used as offices or residential apartments. Experiences from recent earthquakes (EERI, 1985; CSMIP, 1987) have demonstrated the vulnerability of flat-slab to strong ground motion and raised serious concerns on the adequacy of similar construction in this country. In the event of an earthquake, the survival of flat-slab buildings depends largely on the performance of slab-column connections. Slab-column connections in older buildings have a nonductile detail and are typically the most vulnerable link in resisting the lateral loads. Because of the non-ductile reinforcing detail, the moment transfer capacity and shear resistance of connections under earthquake-type loading is expected to be severely limited. Furthermore, the connections lack the ability to prevent progressive collapse of floors after a punching failure has occurred. The lack of such protection resulted in collapse of a number of flat-slab buildings during the 1985 Mexico City earthquake.

The response of flat-slab systems during earthquake-type loading is very much linked to the performance of slab-column connections. Their strength, stiffness, and deformation capacity must be determined first before the overall response of the building can be evaluated. Knowing the deficiencies, it should be possible to develop effective retrofitting schemes to improve the survivability of flat-slab buildings during earthquakes.

1.1 Background

The study is focused on flat-plate buildings which are approximately thirty to fifty years old. The design of flat-plate structures during that time was mostly empirical. Typically, the slab was divided each way into column strips, which served the purpose of beams between the columns; and middle strip, which can be regarded as suspended spans that were carried by column strips. For the simple regular flat-plates, the direct design method was used to calculate moments at critical sections and the total moment was then distributed between column strips and middle strips for the design of the slab.

The shear stress in the connection region was calculated at a section equal to slab thickness minus 1.5 inches away from the column face or outside edge of the column capital parallel to or concentric with the column when no drop panels were used. The shear strength of the critical section was then determined based on the amount of reinforcing bars passing over the column or column capital. If 50% or more of the slab negative reinforcement passed over the column capital, the allowable shear stress was limited to $0.03f_c'$, and if 25% or less of slab reinforcement passed directly over the column capital the allowable shear stress was reduced to $0.025f_c'$. For percentages between 50 and 25, the value was interpolated.

For a concrete compression strength of 3000 psi, the shear strength factors $0.03f_c'$ and $0.025f_c'$ are $1.64\sqrt{f_c'}$ and $1.37\sqrt{f_c'}$, respectively. These values are rather conservative compared with the allowable shear stress of $4.0\sqrt{f_c'}$ in the current building code (ACI 318-89) for a square column. The effect of moment-transfer on connection strength is not accounted for explicitly. The interaction between moment and shear force appears to be reflected through the reinforcing detail; the allowable shear stress is reduced if a smaller percentage of negative moment reinforcement is contained in the column capital. Due to the lower allowable shear stress, some moment-transfer capacity is automatically built into the design. The design of the slab flexural reinforcement was done using the working stress method considering gravity loads only.

Furthermore, the slab reinforcing detail was such that (a) the reinforcing bars were evenly spaced across both column and middle strips, (b) the spacing of reinforcing bars was limited to three times the slab thickness, and (c) the slab positive reinforcement, perpendicular to the slab edge at exterior supports, extended at least six inches into support/column and the slab negative reinforcement was anchored into support with a hook.

The basic procedure for design of flat-slab structures did not change much between 1941 and 1956. However, in 318-56 ACI Building Code, the two allowable values for shear stress were further limited to 100 psi and 85 psi depending upon the amount of slab negative reinforcement passing over the column capital as mentioned earlier. It was about that time that concrete over 4000 psi strength began to be commonly used but no research data was yet available to demonstrate the validity of the existing shear stress limits for this concrete strength. In addition, the minimum length of slab reinforcement for straight and bent-up reinforcing detail were also clarified.

The allowable shear stress was changed to $2.0\sqrt{f_c'}$ in the ACI 318-1963 Building Code. This reflected the consensus that shear strength was proportional to the square root of the concrete compressive strength instead of the direct relationship as described in the previous codes. Furthermore, the shear strength of the connections was to be checked for the most severe of the two conditions: (a) the slab acting as a wide beam unit with a potential diagonal crack extending in a plane across the entire width and, (b) two way action for the slab with potential diagonal cracking along the surface of an inverted truncated cone or pyramid around the column. These changes formed the basis for the shear design contained in the present code. With the recognition of the ultimate strength design, the nominal shear stress for two-way action was calculated by

$$v_u = \frac{V_u}{\varphi b_o d}$$

with critical section located at a distance d/2away from the periphery of column. The reinforcing detail and the overall design procedure, however, remained unchanged.

In the ACI 318-71 Building Code, the maximum allowable shear stress was changed to $4.0\sqrt{f_c'}$ on the critical section and the concept of moment-transfer through eccentricity of the shear about the centroid was recognized. Research at PCA laboratories demonstrated that the allowable shear stress factor of $4.0\sqrt{f_c'}$ was unconservative for rectangular columns and also that 40% of the unbalanced moment at the connection was transferred by eccentricity of the shear varying linearly about the centroid of the critical section. The conclusions regarding the moment transfer were, however, mostly based on tests of square columns. The proportion of moment transferred through shear was calculated by

$$\gamma_v = 1 - \frac{1}{1 + \frac{2}{3}\sqrt{\frac{c_1 + d}{c_2 + d}}}$$

ASCE-ACI Task Committee 426 studied the research data and in 1974 reported on the shear strength of reinforced concrete members and slabs (ASCE-ACI Committee 426, 1974). In this report, the committee recognized the moment-transfer mechanism, the effect of the column aspect ratio on the shear strength, and the column side dimension to slab depth as an important variable which was indirectly taken into account by assuming a pseudo-critical section located at a distance d/2 from the column face. Based on this report, the shear strength calculations were further revised in 1977 ACI 318-77 Building Code as

$$v_u = \varphi\,(2 + \frac{4}{\beta_c})\sqrt{f_c'}$$
$$\leq \varphi 4.0\sqrt{f_c'}$$

The shear design procedure described in 1977 in the ACI 318-77 code remained unchanged in the ACI 318-83 Building Code. The reinforcing detail for connections in flat-slab buildings in seismic zones were made more specific to safeguard against progressive collapse during earthquakes. The shear design of slab-column connections remained mostly unchanged in the ACI 318-89 Building

Code. ACI Committee 352 developed recommendations dealing specifically with the design of connections for earthquake-type loading (ACI Committee 352 1989). These recommendations deviate from the ACI approach in the design of exterior connections and the allowable shear stress is tied to a variety of factors related to connection detail and the loading type. In addition, specific recommendation was made for continuing a certain minimum amount of slab bottom reinforcement through the interior column for protection against progressive collapse.

Research data on the seismic resistance of connections with nonductile detail is almost nonexistent. However, some data is available on the failure of flat-slab buildings under gravity overload situations. In a recent investigation of a collapsed warehouse due to high over loads, Vecchio and Collins (1990) attributed high floor load capacities to membrane action in the slab. It was suggested that the compression of membrane substantially increased both the shear and flexural capacities of the slab.

1.2 Objectives and Scope

The main objective of this investigation was to evaluate the seismic resistance of slab-column connections in non-ductile flat-plate buildings. This included the evaluation of moment-transfer capacity, punching shear strength, stiffness degradation, and lateral drift response of interior and exterior slab-column connections. This study would provide the necessary data for developing retrofit schemes for connections in existing flat-plate buildings to resist moderate earthquakes. Furthermore, the observed moment-rotation response of the connections could form the basis for analytical modelling and predicting the seismic response of existing flat-plate buildings.

Several half-scale two-bay subassemblies consisting of interior and exterior connections will be tested under simulated earthquake-type loading. The two-bay configuration of the test specimens was chosen to allow study of the moment redistribution among connections. The design and reinforcing detail of these subassemblies was based on gravity load design procedures recommended in the building codes of forties and fifties. The scope of the testing program is limited to the study of three variables, namely, the intensity of the gravity loading, configuration of the slab reinforcement, and the presence of the spandrel beams.