SEISMIC EVALUATION AND RETROFITTING TECHNIQUES OF BRIDGE COLUMNS

A DISSERTATION

Submitted in partial fulfilment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With Specialization in Structural Dynamics)

By

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JUNE, 2004

CANDIDATE'S DECLARATION

I hereby certify that work which is being presented in this dissertation entitled, **"SEISMIC EVALUATION AND RETROFITTING TECHNIQUES OF BRIDGE COLUMNS"**, in partial fulfillment of the requirements for the award of the degree of **MASTER OF TECHNOLOGY** in **EARTHQUAKE ENGINEERING** with specialization in **STRUCTURAL DYNAMICS**, submitted in the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee, is an authentic record of my own work carried out for a period from August, 2003 to June, 2004 under the supervision of **Mr. R.N. Dubey**, Assistant Professor and **Dr. Pankaj Agarwal**, Lecturer, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee.

The matter embodied in this dissertation has not been submitted for the award of any other degree or diploma.

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This is to certify that the above statement made by candidate is correct to best of our knowledge.

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ACKNOWLEDGEMENT

I wish to express my sincere regards and gratitude to **Mr. R.N. Dubey**, Assistant Professor, and **Dr. Pankaj Agarwal**, Lecturer, Department of Earthquake Engineering, Indian Institute of Technology Roorkee, Roorkee for their valuable guidance and encouragement during the course of this study.

I express my deep sense of gratitude to **Dr. S. Basu**, Professor and Head, Department of Earthquake Engineering, Institute of Technology Roorkee, Roorkee for providing opportunities to complete this study.

Special thanks to Mr. Rishi Chand, laboratory staff of Quasi-Static Testing laboratory of Department of Earthquake Engineering, for his co-operation in construction of models.

I would like to express sincere regards to my parents and my friends for their support and sacrifices.

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ABSTRACT

Reinforced concrete bridge columns constructed before mid 1970's are vulnerable to earthquake forces because of lack of ductile detailing provisions. The vulnerability of these columns has been emphasized in San Fernando, Northridge, Kobe, Whittier and some other earthquakes all over the world. These columns have failed in flexural, shear and bond. In order to decide the strategy to retrofit these bridge columns, it is necessary to carry out seismic evaluation for determining seismic capacity, weaker sections and failure modes. In this study an attempt has been made to compare the load carrying capacity and ductility of columns before and after retrofitting. Two columns are constructed and tested in quasi-static test facility under cyclic loading. Two columns are designed as per IS: 456-2000 with special confining reinforcement and its lateral load carrying performance has been studied under cyclic loading. One of these columns has been retrofitted using steel jacketing and other by glass fiber reinforced polymer (GFRP) to increase their lateral confinement. These retrofitted columns have been tested again using similar loading condition. Test data are presented and comparison has been made for their behavior before and after retrofitting. Results of this study indicate that significant improvement in ductility and energy absorption capacity can be achieved after retrofitting.

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INTRODUCTION

1.1 Background

Earthquakes have a habit of identifying structural weaknesses and concentrating damage at these locations. Building structures have high degree of redundancy compared to bridges, which have a little or no redundancy. Hence failure of one part of structural element may likely result in collapse of the entire bridge system than in the case of building. The destructive earthquakes all over the world in last three decades have caused vast damage to the highway bridge columns. Investigations indicated that the bridges designed and constructed prior to the development of modern seismic design guidelines are vulnerable to disastrous or severe damage due to many potential structural problems.

The most seismically vulnerable aspect of a bridge system is the column. Reinforced concrete bridge columns constructed before the mid 1970's have many characteristics that make them vulnerable to the effects of severe earthquake loading. The commonly seen deficiencies of damaged bridge columns may be characterized as (i) insufficient shear strength or ductility, (ii) inadequate anchorage or bonding and (iii) insufficient flexural strength or ductility. Longitudinal reinforcement commonly is lap- spliced just above the footing level causing undependable flexural capacity. The provided lap length and confinement by transverse reinforcement often are inadequate for ensuring that the reinforcement can develop and sustain the yield stress under earthquake loading. Column transverse reinforcement typically is poorly configured and widely spaced, resulting in inadequate confinement of the longitudinal reinforcement and column core for demands related to axial load, flexure and shear.

The consequences of these inadequacies ranging from severe damage to complete bridge collapse are evident in the earthquake reconnaissance literature. In order to upgrade the existing bridge columns built in pre-1971 period to meet the current seismic design regulations in regions with high seismicity, various retrofitting methods have been developed by researchers and practice engineers. Steel jacketing, concrete jacketing, advanced composite wrapping have been proven to be effective measures to retrofit bridge columns for increase strength and ductility. In this study steel and GFRP jacketing has considered.

1.2 Objectives

The objectives of the present study are as follows:

i. To study the behavior of typical reinforced concrete bridge columns under cyclic lateral loading.

ii. To study the response of damaged bridge columns retrofitted with steel and GFRP jacket materials.

iii. To compare the hysteretic behavior of bridge columns before and after retrofitting.

1.3 Scope of Investigation

In the framework of this dissertation, two models of R.C. bridge columns have been studied under cyclic loading in Quasi-static test facility with an aim to evaluate the failures by measuring modal parameters of columns. The reinforcement detailing of these two models are detailed as per IS 456: 2000 with special confining reinforcement using IS 13920: 1993 code.

The models have been planned, constructed and fixed on the strong floor. One end of the hydraulic actuator is attached to the reaction wall and the other end is fixed to the top of model to apply the lateral loads by means of plates and bolts. The models are subjected to alternate cyclic loading in the form of sine wave in a quasi-static test. The models have been tested to ultimate failure and their deficiencies are studied to evaluate the retrofit methods. The models are retrofitted with appropriate techniques and tested again to ultimate failure. The results of this investigation before and after retrofitting of columns are compared.

1.4 Organization of the Thesis

This thesis consists of five chapters.

Chapter-1: Introduces the background, objectives and scope of work.

Chapter-2: This chapter presents typical structural deficiencies of early RC bridge columns, review of research towards retrofitting, Quasi-static testing technique and past experimental work has been included.

Chapter-3: This chapter presents the design, detailing and fabrication of circular bridge column models. The scheme and method of testing have also been discussed.

Chapter-4: Explains the seismic retrofitting techniques and procedures adopted models after retrofitting.

Chapter-5: This chapter presents results and discussions of the experimental work.

Chapter-6: This chapter discusses about the summary and conclusions drawn from this study.

LITERATURE REVIEW

2.1 General

Investigations during last three decades indicated that bridges designed and constructed prior to the development of modern seismic design guidelines are vulnerable to disastrous collapse or severe damage due to many potential structural problems. A study of the damage sustained by engineering structures in past earthquakes provides one of the best means of evaluating the seismic resistance of various types of structures and serves as the ultimate test for assessing the adequacy of seismic design procedures. In order to study the seismic performances of reinforced concrete bridge columns it is very much necessary to have an idea of the behavior of columns in past earthquakes, different causes of failure and various modes of failure. In this chapter typical structural deficiency of early RC bridge columns, review of research towards retrofitting, Quasi-static testing technique and past experimental work has been included.

2.2 Typical Structural Deficiencies of Early RC bridge Columns

a) Inadequate flexural strength of members, due to insufficient longitudinal reinforcement. However, this was found to be not normally a problem for bridge columns, particularly if the reinforcement had been designed using elastic theory (working stress) design. Elastic theory design for column sections was conservative resulting in eccentrically loaded columns with an actual flexural strength, which is higher than expected.

b) Inadequate ductility and shear strength of potential plastic hinge regions of members, due to insufficient transverse reinforcement to provide the required confinement of compressed concrete, restraint against lateral buckling of longitudinal reinforcement and shear resistance.

c) Inadequate anchorage of longitudinal bars, due to insufficient development length to maintain the yield strength during cyclic loading. Also, the presence of lapped longitudinal bars in potential plastic hinge regions, which means that yielding may concentrate over small lengths of bar outside the lap and slip bars may occur at top. Early structures used plain round bars, rather than deformed bars, which increased the risk of bond failure.

d) Inadequate anchorage of transverse reinforcement, due to being lapped and not welded in cover concrete or not properly anchored by bending around the longitudinal bars.

2.3 Review of Research

The collapse and severe damage to many buildings and bridges in recent earthquakes have highlighted the need for the seismic retrofit of seismically insufficient structures. RC columns, being the key lateral and vertical load-resisting members in RC structures, are particularly vulnerable to failures in earthquakes, so their retrofit is often the key to a successful seismic retrofit strategy. As a result, many retrofit projects of building and bridge columns using steel and FRP composites have been carried out all over the world, some of them are listed below. The use of steel jacketing as external confinement is based on extension of the confined concrete model first developed by Mander, Priestly and Park (1988). The earliest paper presented in the bibliography pertaining to composites is by Katsumata, Kobatake and Takeda (1988).

Saadatmanesh, et al. (1997) investigated the behavior of typical rectangular bridge columns with substandard design details for seismic forces. A method utilizing fiber reinforced polymer composites to retrofit existing bridge columns is investigated in this paper. Five rectangular columns with different reinforcement details were constructed and tested under reversed cyclic loading. Two columns were not retrofitted and were used as control specimens so that their hysteresis response could be compared with those retrofitted columns.

Ma and Xiao (1999) have done experimental studies on seismic retrofit and repair of typical circular bridge columns with poor lap splice details utilizing glass fiber reinforced polymer (FRP) composite jackets and epoxy. A total of seven tests on three half-scale model columns were conducted. In this one column was tested under as-built condition and other two were retrofitted with composite material. Based on this study they have concluded that dramatic improvement in ductility and energy absorption capacity of columns can be achieved using retrofit and repair methods.

Jirsa, et al. (1996) has described an experimental research program on the use of steel jackets for seismic retrofit on non-ductile reinforced concrete frame columns. Eleven large-scale columns were tested to examine the effectiveness of various types of steel jackets for improving the ductility and strength of columns with an adequate lap splice in the longitudinal reinforcement. Response of the columns before and after being strengthened with steel jackets was examined. In this paper design guidelines for the use of rectangular steel jackets as a seismic retrofit for non-ductile reinforced concrete columns are presented.

Priestly, et al. (1994) has presented theoretical and experimental investigation conducted to study the shear failure mode of reinforced concrete bridge columns designed before 1971, and to establish the effectiveness of full- height steel jackets for enhancing the seismic shear strength, is described. This investigation was divided into two papers, in first part theoretical considerations relating to assessing the shear strength of existing columns of circular or rectangular sections are presented. In part two assessment and retrofit of bridge columns to shear failure, experimental results are presented from two test programs. Based on this study they concluded that jacketed columns performed extremely well, with stable hysteresis loops being achieved to displacement ductility levels of $\mu_\Delta>8$.

2.4 Past Experimental Work

2.4.1 Steel jacketing

In the study described, two columns of a 40% dimensional scale were constructed and tested under axial load and reversed cyclic loading. The columns were designed so that flexural failure would precede brittle shear failure, for this the columns are designed as six times as high as the diameter. The transverse steel ratio was slightly different than that in the prototype, 0.118%. The load tests consisted of first applying the axial load to the column via two high-strength steel bars post-tensioned to the laboratory floor with center hole jacks. The horizontal force was applied with a double acting actuator having a push capacity of 667 kN, a pull capacity of 578 kN and a maximum stroke of 455mm.All the columns were tested under the following load pattern. Two cycles to ± 36 kN and one cycle to ± 67 kN were imposed in order to verify operation of the data acquisition system and to look for cracks that may develop before this load level. Five cycles to ± 122kN, approximately 50% of the lateral load corresponding to the nominal flexural strength, were imposed to check for premature lap splice bond failure. One cycle to ± 178 kN, the lateral load which approximately corresponds to that required to cause first yield, was carried out to define the experimental yield displacement. The average of the displacements under the push and pull cycles was defined as the yield displacement.

The steel jacket had a thickness of 5 mm with a 6 mm grout infill. Styrofoam is placed on the column surface prior to installation of the steel jacket. The behavior of columns is presented as follows.

Column 1 experienced flexural cracking during the first cycle to 67 kN and spread to half of the column height when the force was increased to 122 kN. Creaks outside the lap splice region were wider and grew faster than those in the lap splice region. Vertical cracks developed near the base at 178 kN, indicating the beginnings of bond failure. Spalling began at μ_Δ=1 and become extensive at μ_Δ= 1.5. Final failure was due to the loss of confinement of the lap splice region caused by extensive spalling and stirrup fracture at μ_Δ= 4. Retrofit column 2 presented some difficulties in observing crack formation because an epoxy resin seal at to and bottom of the jacket, in addition to the jacket itself, obscured much of the critical areas. Slightly inclined flexural cracks were noted above the jacket at a load level of 178 kN indicating the influence of shear on the crack formation. Separation of column and jacket was noticed at this stage. The flexural cracks were essentially spaced with the transverse hoops, indicating that the stirrups were initiating the cracking. The summary of the tests indicates that for "As-built" columns with widely spaced transverse reinforcing and short lap splices typical of pre-1971 designs, the columns are unlikely to reach their nominal flexural capacity and will degrade rapidly. These tests also indicate that failure of column takes place if the column is retrofit and the footing is weak. The tests also showed that the steel jacket technique is effective as a repair method for damaged columns. (Wipf, et al. 1997).

2.4.2 Composite Jacketing

A method utilizing fiber reinforced polymer composites to retrofit existing bridge columns is investigated in this paper. High-strength FRP straps are wrapped around the column in the potential plastic hinge region to increase confinement and to improve the behavior under seismic forces. Five rectangular, reinforced concrete bridge column-footing assemblages were designed with a scale factor 1/5 that of prototype bridge columns. The test specimens were designed to model typical pre-1971 design of existing highway bridge columns in a zone of high seismic risk. Each specimen modeled a prototype single column bent to emphasize two problems: (1) inadequate starter bar lap length; (2) insufficient transverse reinforcement. The composite strap was applied only in the potential plastic hinge region estimated to be twice that of the effective depth of the cross section; i.e., in the 635 mm long portion of the column above the top face of the footing.

The setup was designed for testing column-footing assemblage subjected to lateral loading. The specimens were tested in a steel reaction frame. Two independent loading systems were used to apply the load to the specimens as shown in Fig.5.2. The axial load of 445 kN was applied to the column by prestressing of 25 mm diameter high-strength steel rods against the base beams of the test frame, which was strongly bolted to the 915 mm thick concrete floor, before applying the lateral loads to the specimens. The actuator was capable of moving the top of the specimen 127 mm in both positive and negative directions. A displacement of 127 mm corresponds to a ratio of lateral displacement to column height of approximately 7%. Results of two columns with details as column 1 with starter bars, longitudinal steel ratio 2.7%, and column 2 with 2.7% longitudinal steel ratio and with rectangular confining strap configuration are compared.

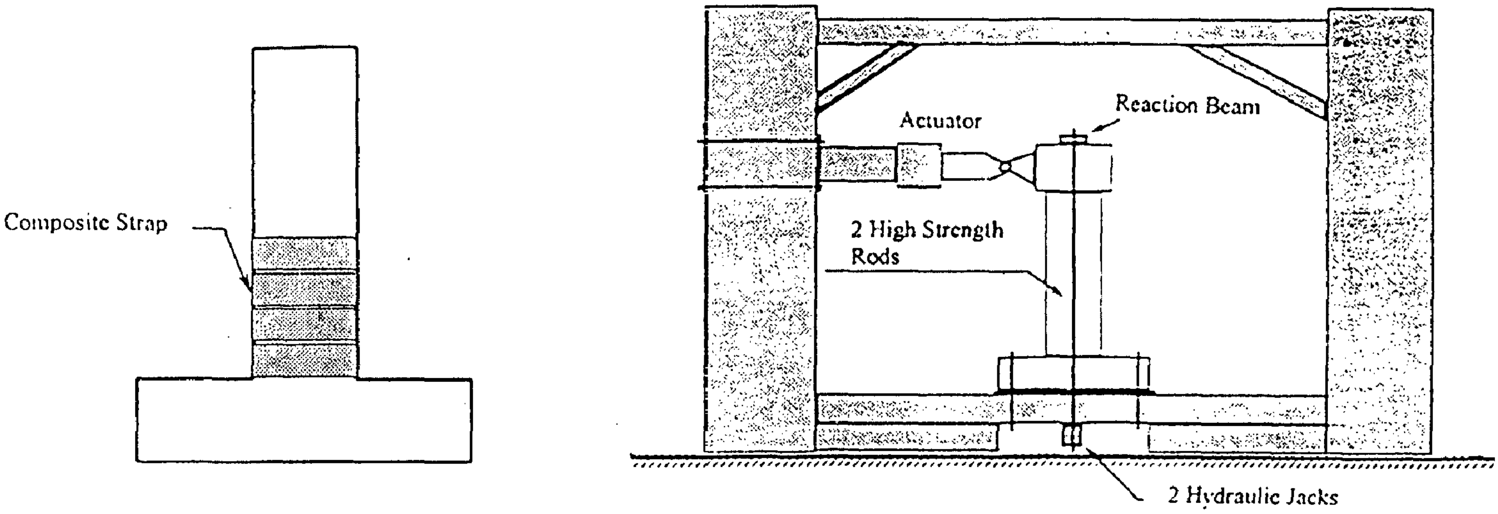


FIG. 2.1 Typical Concrete Column Wrapped with Composite Strap in the Potential Plastic Hinge Region and Test Set up

Column1 experienced rapid degradation in strength occurred early and with very narrow energy dissipation loops. The lateral strength of column1 started to drop quickly at the push cycle to μ_Δ= 1.5. Because of bond failure in the lap spliced bars within the plastic hinge region of the column, the lateral load carrying capacity was reduced approximately by 80% of the calculated value at the ductility level of μ_Δ= 4. Column2 retrofitted with eight plies of FRP composite straps resulting in a total strap thickness of 6mm and an active confinement scheme showed substantial improvement in the lateral load-displacement response as compared to column1. The presence of the confining strap prevented premature bond failure in the splice region. Consequently, the longitudinal bars could undergo significant plastic deformation, resulting in a larger lateral load and displacement capacities. It is noted, however, that the additional flexural capacity as a result of confinement with composites straps, could potentially lead to premature shear failure in the unwrapped portion of the column. If this is the case, the region outside the plastic hinge region must be wrapped with adequate number of plies to develop the required shear capacity. The lateral load exceeded the predicted capacity of the unretrofitted column at the first cycle to μ_Δ= 1.5. The column continued to resist more lateral loads up to the ductility level μ_Δ= ± 5 with very stable hysteresis loops.

A slight decrease in the column strength was noted at the ductility level of μ_Δ= 6, which appeared to be due to small slippage between the main longitudinal reinforcement and the lapped starter bars. (Saadatmanesh, et al. 1997).

2.5 Quasi-Static Testing Technique

The term quasi-static implies that, testing is conducted at a sufficiently slow rate such that strain-rate effects are insignificant. It is most economical and most common method for obtaining information on the inelastic behavior of structure in which prescribed history of load or displacement are imposed on structural system. Such prescribed displacement histories can be particularly valuable in (i) Assessing the effects of different structural details on the inelastic behavior of structures by subjecting different specimens to identical deformation histories and (ii) Studying the basic mechanism that affects the inelastic behavior of a particular structure by varying the magnitude, rate or pattern of the applied deformat5ion histories. This type of test provides the reversing character of the loading that distinguished dynamic response from response to unidirectional static loading. In addition, as the load application points are fixed, the moments and shears are always in phase; a condition, which does not generally occur in dynamic response.

2.5.1 History of Quasi-static testing

In development of structural engineering experimental testing has been a cornerstone since 17th century. Experimental testing has accompanied development of different analytical formulations.

a) Verifying new concepts in structural engineering

b) Studying failure mechanism of various systems.

c) Studying design and detailing provisions of codes and make improvement in these provisions.

Around 1815 in France, A. Duleau on iron bridges and P.C. Dupin on wooden ships started first structural experimental research work. Invention of screw type testing machine in 1850's and hydraulic testing machines in 1920's as well as the development of new measurement instrumentation gave a new direction in understanding the material behavior. This development led to study of combination of material and structural behavior.

But research on seismic response of structural elements started only after 1906 San Francisco and 1923 Kanto earthquake. Initially research was centered on understanding dynamics of full-scale structures (Blume, 1935). Development of plastic design and improvement in analysis for frame action led to testing of full-scale machine for testing cyclic behavior of connections. They equipped the testing machine with wing frames to form a self-reacting system and inverted the position of the specimen in each half cycle. This revealed the characteristic degradation of strength and stiffness of connections, which is still the focal point of modern seismic testing (Leon and Deierlein, 1996).

Advances in hardware have improved the ability of quasi-static testing for studying behavior of structures under cyclic loads. These recent advances include:

a) Closed-loop servo controlled hydraulic systems, which allow greater precision in applying the loads or displacements.

b) New transducers, which allow precision measuring of structural responses.

c) Test facilities for testing of full-scale and prototype.

2.5.2 Merits and Limitations

The primary advantage of quasi-static test as compared to other methods of testing is its economy and ease of interpretation of results. The expenses incurred while testing of one building model in shake table test could be more than entire series of quasi=static subassembly tests (Leon and Deierlein). Also quasi-static can be performed with the available laboratory equipment and techniques without any specialized training. Some other advantages of quasi-static test are:

a) Full-scale subassemblies and prototypes could be tested. It is important for seismic events, as the materials do not follow similitude rules.

b) Different limit states like cracking, yielding, and fracture can be carefully observed at different states of testing.

c) Helps in the development of load deformation behavioral model. It is important as strain rate and damping property of different material is not well known and cannot be included in the analysis.

d) Allows number of tests at low costs by varying different parameters as grade of concrete, steel and different detailing patters.

Limitations of quasi-static testing method are as follows:

a) Under earthquake loading if strain rate effects are significant, quasi-static test provides misleading or incorrect results.

b) As the load is applied in a predetermined pattern, it provides inaccurate result if the element behavior is sensitive to bending or shear and bending or axial force ratios.

c) If ductility and energy dissipation capacity is important, then the quasi-static test results can be safely assumed as lower bound.

d) Inertia and damping effects are not simulated accurately.

DESIGN OF SPECIMEN AND FABRICATION OF TEST SETUP

3.1 Introduction

The experimental testing was carried out in two phases. The first phase involved testing of two columns constructed keeping in view the capacity of test set up, to evaluate the strength, ductility, and failure mechanisms. Two columns are planned and constructed with same detailing; one is for steel jacketing and other for FRP jacketing. The second phase included testing of these two columns after employing different retrofit schemes i.e., steel and FRP jacketing. Damage detection of 'as built' bridge columns can be evaluated through Quasi-static testing subjected to alternate cyclic loading. The loading pattern and history must be carefully chosen to be general enough to provide the full range of deformation that the structure will experience under the earthquake excitation. This chapter presents first phase.

3.2 Description of the Models

The dimensions of the modeled circular bridge columns are as follows

Materials used are M_{20} concrete and Fe 415 steel.

Diameter of the columns = 300 mm

Height of the models = 1500 mm

Height of the application = 1200 mm

3.3 Codal provisions

3.3.1 IS Code: 456-2000 Specifications

a) Requirements governing Reinforcement and Detailing:

- Reinforcing steel of same type and grade shall be used as main reinforcement in a structural member. However, simultaneous use of two different types or grades of steel for main and secondary reinforcement respectively is permissible.
- The recommendations for detailing for earthquake-resistant construction given in IS 13920 should be taken into consideration.

b) Requirements of Reinforcement for columns:

- Longitudinal reinforcement: a) The cross sectional area of longitudinal reinforcement, shall be not less than 0.8 percent nor more than 6 percent of the gross cross sectional area of the column. b) In any column that has a larger cross- sectional area than that required to support the load, the minimum percentage of steel shall be based upon the area of concrete required to resist the direct stress and not upon the actual area. c) The minimum number of longitudinal bars provided in a column shall be four in rectangular columns and six in circular columns. d) The bars shall not be less than 12 mm in diameter. e) Spacing of longitudinal bars measured along the periphery of the column shall not exceed 300 mm.
- Transverse reinforcement: a) A reinforced concrete compression member shall have transverse or helical reinforcement so disposed that every longitudinal bar nearest to the compression face has effective lateral support against buckling. The effective lateral support is given by transverse reinforcement either in the form of circular rings capable of taking up circumferential tension or by polygonal links with the internal angles not exceeding 135^0. The ends of the transverse

reinforcement shall be properly anchored. b) Arrangement of transverse reinforcement: 1) If the longitudinal bars are not spaced more than 75 mm on either side, transverse reinforcement need only to go round corner and alternate bars for the purpose of providing effective lateral supports. 2) If the longitudinal bars spaced at a distance of not exceeding 48 times the diameter of the tie are effectively tied in two directions, additional longitudinal bars in between these bars need to be tied in one direction by open ties. 3) Where the longitudinal reinforcing bars in a compression member are placed in more than one row, effective lateral support to the longitudinal bars in the inner rows may be assume to have been provided if: i) transverse reinforcement is provided for the outer-most row in accordance with ii) no bar of the inner row is closer to the nearest compression face than three times the diameter of the largest bar in the inner row.4) Where the longitudinal bars in a compression member are grouped and each group adequately tied, the transverse reinforcement for the compression member as a whole may be provided on the assumption that each group is a single longitudinal bar for purpose of determining the pitch and diameter of the transverse reinforcement. The diameter of such transverse reinforcement need not, however, exceed 20 mm.

3.3.2 IS Code: 13920-1993 Specifications

This standard covers the requirements for designing and detailing of monolithic reinforced concrete structures so as to give them adequate toughness and ductility to resist severe earthquake shocks without collapse. Provisions of this code shall be adopted in all reinforced concrete structures, which satisfy one of the following four conditions.

a) The structure is located in seismic zone IV or V;
b) The structure is located in seismic zone III and has the importance factor greater than 1.0;
c) The structure is located in seismic zone III and is an industrial structure; and
d) The structure is located in seismic zone III and is more than storey high.

The minimum grade of concrete and the maximum grade of steel as specified by this code are M_{20} concrete and Fe 415 respectively. Some of the important guidelines of this code for reinforced concrete column are provided here

Transverse Reinforcement:

a) Transverse reinforcement for circular columns shall consist of spiral or circular hoops. In rectangular columns, rectangular hoops may be used. A rectangular hoop is a closed stirrup, having a 135^0 hook with a 10-diameter extension (but not <75 mm) at each end that is embedded in the confined core as shown in Figure 3.1.
b) The parallel legs of rectangular hoops shall be spaced not more than 300 mm centre to centre. If the length of any side of the hoop exceeds 300 mm, a crosstie shall be provided. Alternatively, a pair of overlapping hoops may be provided within the column. The hooks shall enlarge peripheral longitudinal bars.
c) The spacing of hoops shall not exceed half the least lateral dimension of the column, except where special confining reinforcement is provided.
d) The design shear force for columns shall be the maximum of:
   I. calculated factored shear force as per analysis, and

II. factored shear force given by

$$V_u = 1.4\left[\frac{M_{u,\lim}^{bL} + M_{u,\lim}^{bR}}{h_{st}}\right]$$

Where $M_{u,\lim}^{bL}$ and $M_{u,\lim}^{bR}$ are moment of resistance, of opposite sign, of beams framing into column from opposite faces and h_{st} is the storey height as shown in Figure 3.2. The beam moment capacity is to be calculated as per IS 456: 2000.

Special confining Reinforcement:

This requirement shall be met with, unless a larger amount of transverse reinforcement is required from shear strength considerations.

a) Special confining reinforcement shall be provided over a length l_0 from each joint face, towards midspan, and on either side of any section, where flexural yielding may occur under the effect of earthquake forces as shown in Figure 3.3. The length l_0 shall not be less than a) larger lateral dimension of the member at the section where yielding occurs, b) $1/6^{th}$ of clear span of the member, and c) 450 mm.

b) When a column terminates into a footing or mat, special confining reinforcement shall extend at least 300 mm into the footing or mat.

c) When the calculated point of contraflexure, under the effect of gravity and earthquake loads, is not within the middle half of the member clear height, special confining reinforcement shall be provided over the full height of the column.

d) The spacing of hoops used as special confining reinforcement shall not exceed 1/4 of minimum member dimension but need not be less than 75 mm nor more than 100 mm.

e) The area of cross section A_{sh} of the bar forming circular hoops or spiral, to be used as special confining reinforcement, shall not be less than

$$A_{sh} = 0.09 S D_K \frac{f_{CK}}{f_y}\left[\frac{A_g}{A_K} - 1.0\right]$$

where A_{sh} = area of the bar cross section,

S = pitch of spiral or spacing of hoops,

D_K = diameter of core measured to the outside of the spiral or hoop,

f_{CK} = characteristic compressive strength of concrete cube,

f_y = yield stress of steel

A_g = gross area of the column cross section and

A_K = area of the concrete core

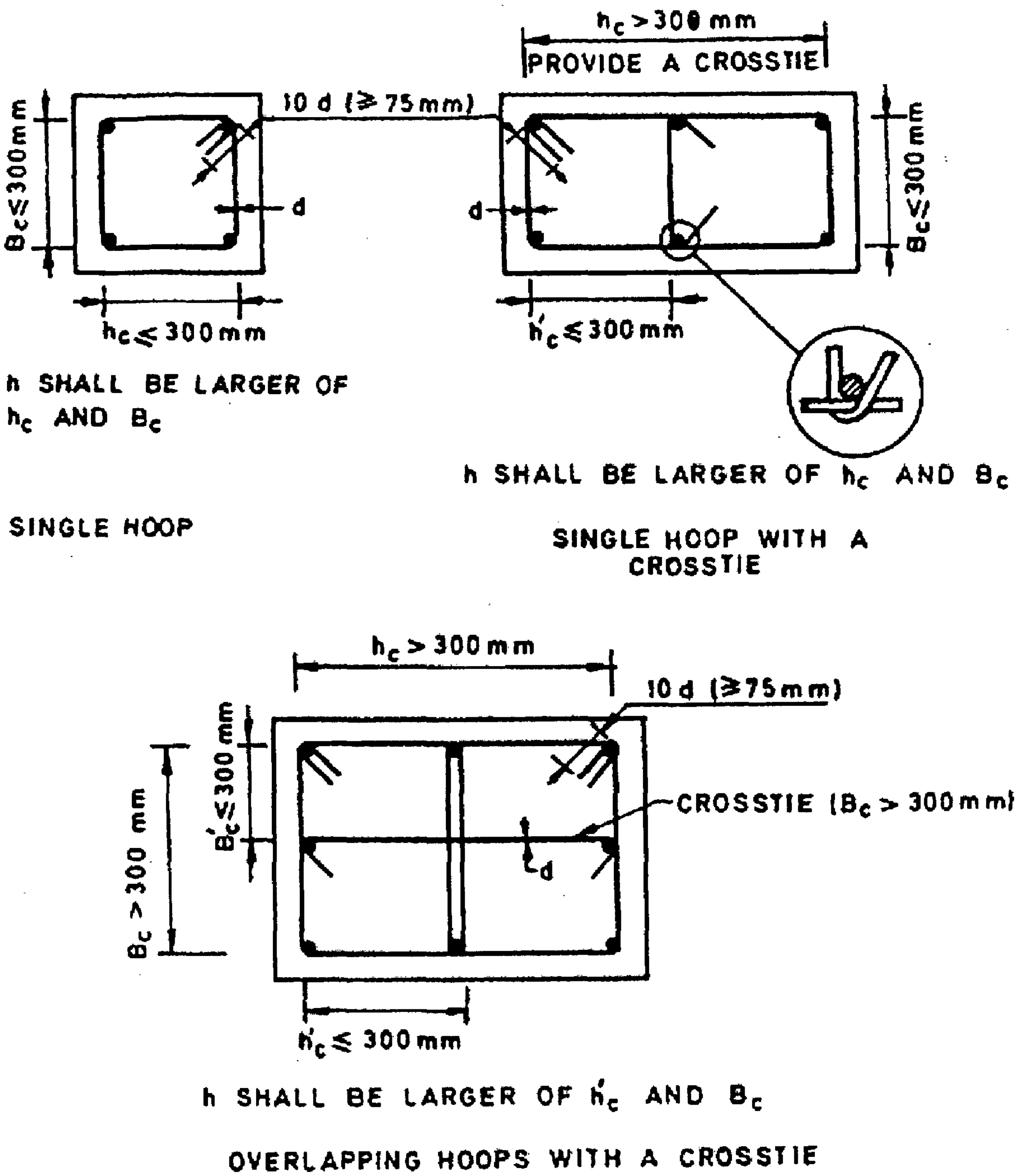
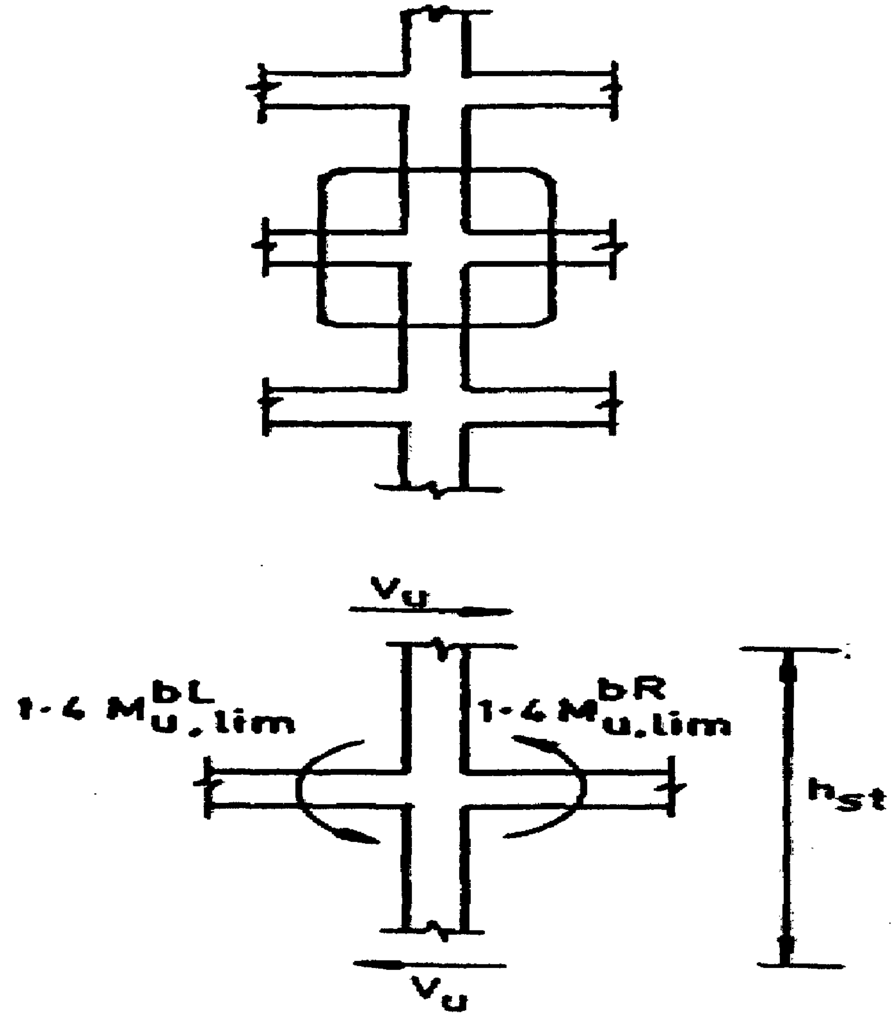


Fig. 3.1 Transverse Reinforcement in Column



$$V_u = 1.4\left(\frac{M_{u,lim}^{bL} + M_{u,lim}^{bR}}{h_{st}}\right)$$

Fig. 3.2 Calculation of Design Shear Force for Column

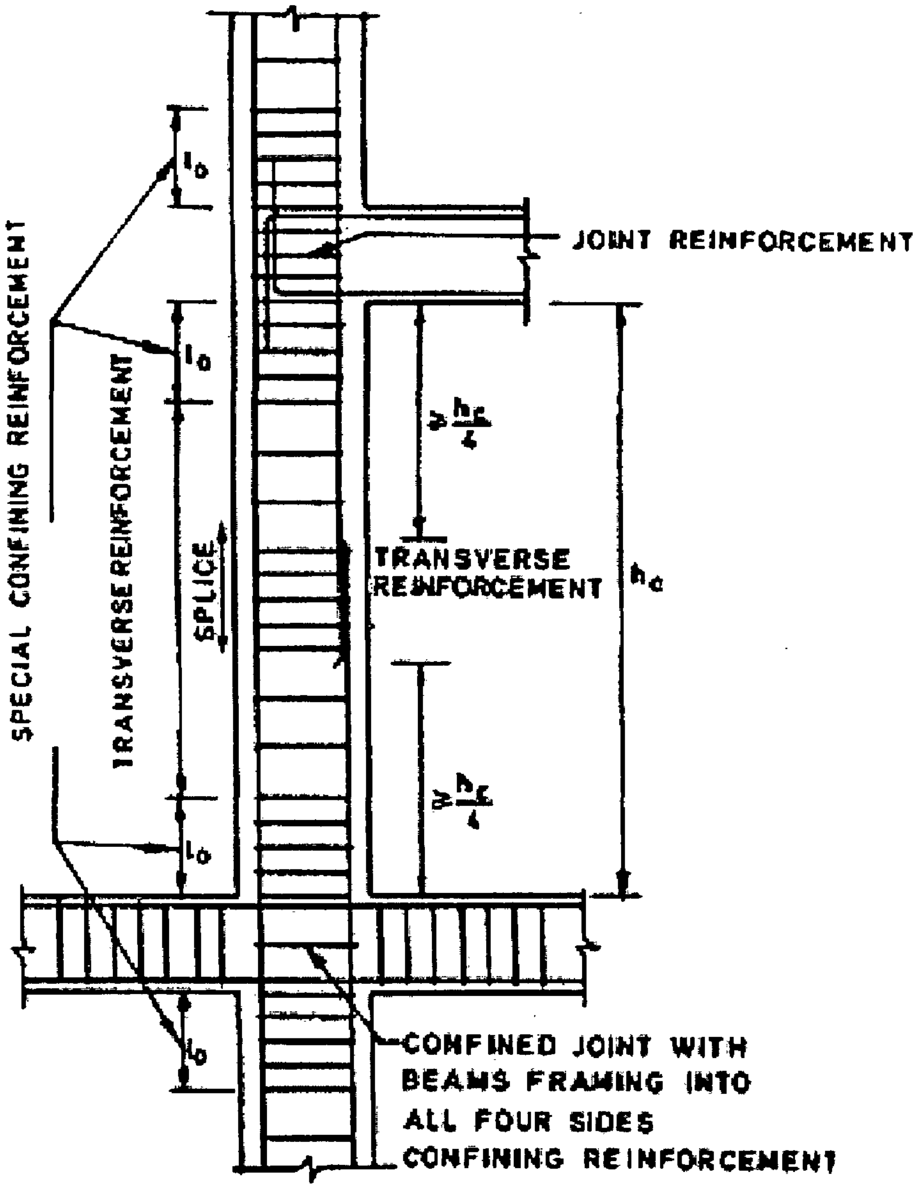


Fig. 3.3 Column and Joint Detailing

3.4 Design and Construction of the Models

3.4.1 Design of the Models

The two models were constructed with same longitudinal reinforcement, height, cross-section and material. Models were constructed as per IS: 456-2000 with special confining reinforcement and 3% steel was provided for longitudinal reinforcement. The design has been done by keeping an eye on the limitation of facility available in Quasi-static lab. The columns were designed for lateral load of 40 kN. Nominal cover of 40 mm for column and 50 mm for footing has been provided. Area of longitudinal reinforcement has found to be 2167 mm^2 and plastic hinge length as 292 mm before retrofitting. In both models 12 number of 16 mm bars were used as longitudinal reinforcement.

3.4.2 Detailing of the models

Special confining reinforcement is provided according to the provisions of the code IS: 13920-1993. Total area of confining steel is1414 mm^2, in which 10 mm bars were provided at a spacing of 95 mm c/c as special confining reinforcement over the length of 450 mm from bottom and rest portion of the section was provided with same bars @ 150 mm c/c. Reinforcement detailing of models has been shown in Figure.3.4 and the detailing provisions are applied as shown in Plate 3.1.

3.4.3 Materials Used

In construction of models 20 mm size and angular shape coarse aggregate and fine sand from Solani River was used. The Indian Standard code IS: 10262-1982 was adopted for mix design to gain M_{20} strength by using Grade-43 cement. The proportions of the mix obtained were 1:1.5:3 with water cement ratio of 0.5.Tor steel bars of 16 mm diameter with yield strength 415 N/mm^2 were used as longitudinal reinforcement of the models. For both models 10 mm diameter bars were used as transverse reinforcement.

3.4.4 Casting of Specimen

Test specimen was casted and then shifted to test position after curing. After that a steel plate of 1.5m x 1.5m is fixed to the floor by bolts of size 18mm (3/4 inch). The foundation reinforcement was welded to this plate. The formwork for footing has done by bricks to get perfect circular and for column it has done by thin steel plate as shown in Plate 3.2. The bolts connecting the model and actuator were welded to the column reinforcement 30 cm from the top. Shuttering was made and concreting was done. The same material and workmanship were conducted for both models. The test specimen was cured for 28 days. Plates 3.3 and 3.4 show the Models M1 and M2 after casting has completed.

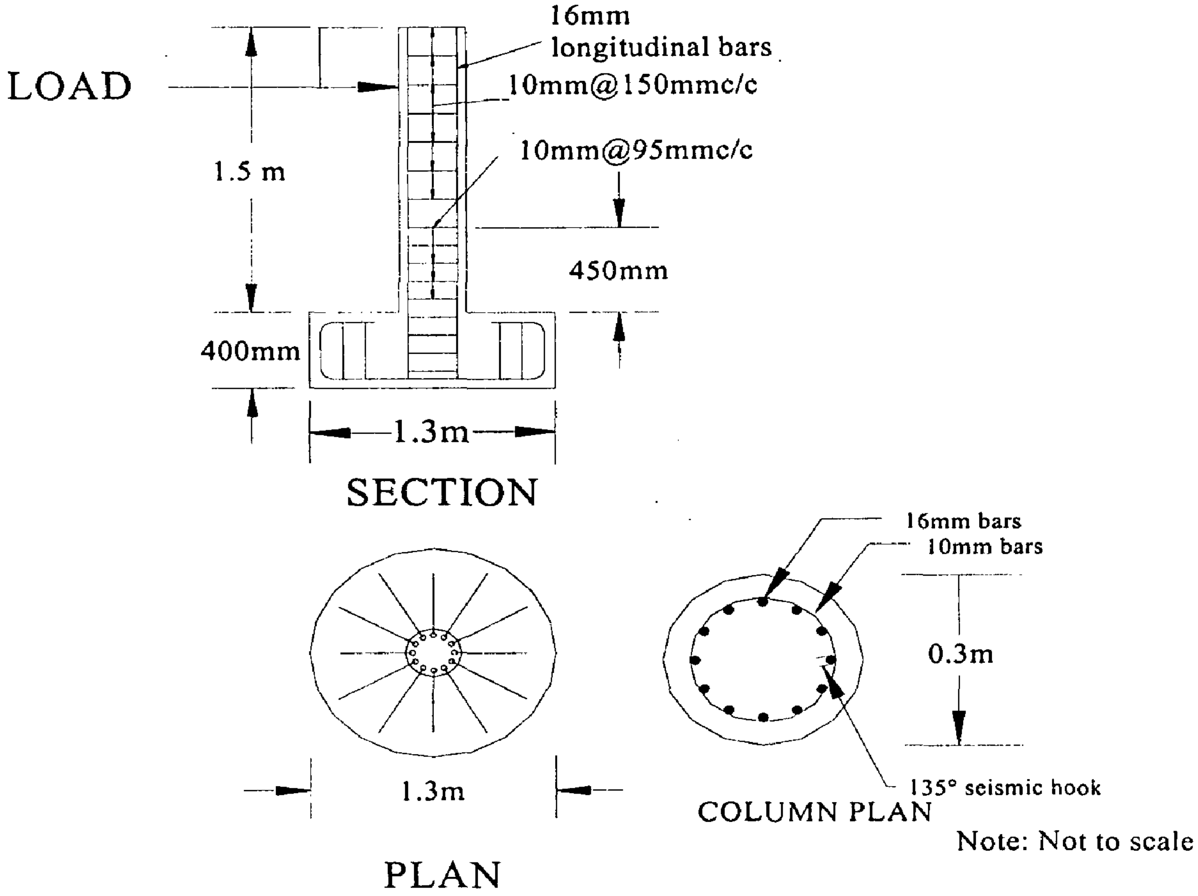


Fig.3.4 Reinforcement Detailing of Models M1 and M2

3.5 Quasi-Static Testing

3.5.1 Components of Quasi-static Testing facility

Quasi-static test facility consists of following components:

Reaction wall and strong floor

In quasi-static laboratory, two reaction walls of height 3.0 m and thickness 0.6 m perpendicular to each other are there to facilitate bi-directional load applications. They were designed for a base shear of 150 kN and a bending moment of 300 kN-m. They are provided with holes spaced at 25 cm c/c in order to fix the actuator at any elevation. The supporting floor is of reinforced concrete of thickness 1.0 m provided with bolts for anchoring the test specimen at the base firmly.

Hydraulic Equipment

The hydraulic equipment consists of hydraulic actuators having a capacity of 100 kN(static) with maximum permissible stroke of 300 mm mounted with pedestal base, swivel forces developed are measured by displacement swivel head and LVDT assembly. The displacement imposed on a structure and the restoring forces developed are measured by displacement and load transducer respectively. These are electronic devices that correlate the displacement and force variations with voltage changes. A perfect device gives an exact linear correlation. The accuracy of these voltage changes depends on their qualities, the calibration techniques and the installations.

The displacement of a structure is controlled by means of actuator-controlled system during a test. An electronic servo-controller is used to command the displacement of a hydraulic actuator in response to the difference between the command signal and the measured displacement. The response of an actuator-controlled system to a displacement signal from a computer depends on the quality and flow capacity of servo valve, which drives the hydraulic actuator and the gain setting on the controller. If the gain is too high, the system may become unstable and the actuator will overshoot and oscillate about the command displacement. Therefore, an optimal gain should be selected if an actuator is to respond sensitively and is stable to a command signal. The maximum response speed of an actuator is limited by the capacity of the servo-valve, which is specified in terms of gallons of fluid flow per minute. In general, the selection of servo-valve capacity depends on the size of the actuator and velocity requirement.

A hydraulic power supply of 90 lpm has been used for supplying oil to actuators. It consists of two separate motor pump units, drawing oil from a common reservoir and delivery to a common pressure supply line. A cooling tower has been used for controlling the temperature of oil.

Data Acquisition System

M-9500 controller has been used for controlling the movement of actuators and for data acquisition. The data acquisition unit is an A/D converter that translates analog feedback from measurement instrument to digital signal, which is returned to the main computer. This instrument has a resolution limit. In addition to the resolution errors, random electrical noise is usually inevitable in this system. The data can be processed through a personal computer connected with controlling tower as shown in Plate 3.5.

3.5.2 Test Setup

The steel plate with model was anchored to the strong floor with the foundation bolts of 18mm diameter. One end of the actuator was fixed to the reaction wall and the swivel head portion was fixed to the model. The actuator was connected horizontally to the model to prevent torsional moments as shown in Plates 3.3 and 3.7

3.5.3 Testing procedure

The test units were brought to the position and tied down to the reaction floor as shown in Figure 3.5. Horizontal load was applied to the top of the models at a height of 1200 mm from footing by servo-controlled hydraulic actuator. Numbers of input files were created in the computer for generating the load history. Loading history was chosen such that it could capture the critical issues of component capacity as well as seismic demand. In the beginning displacement is applied in displacement mode and loading cycles applied were symmetric sinusoidal wave.

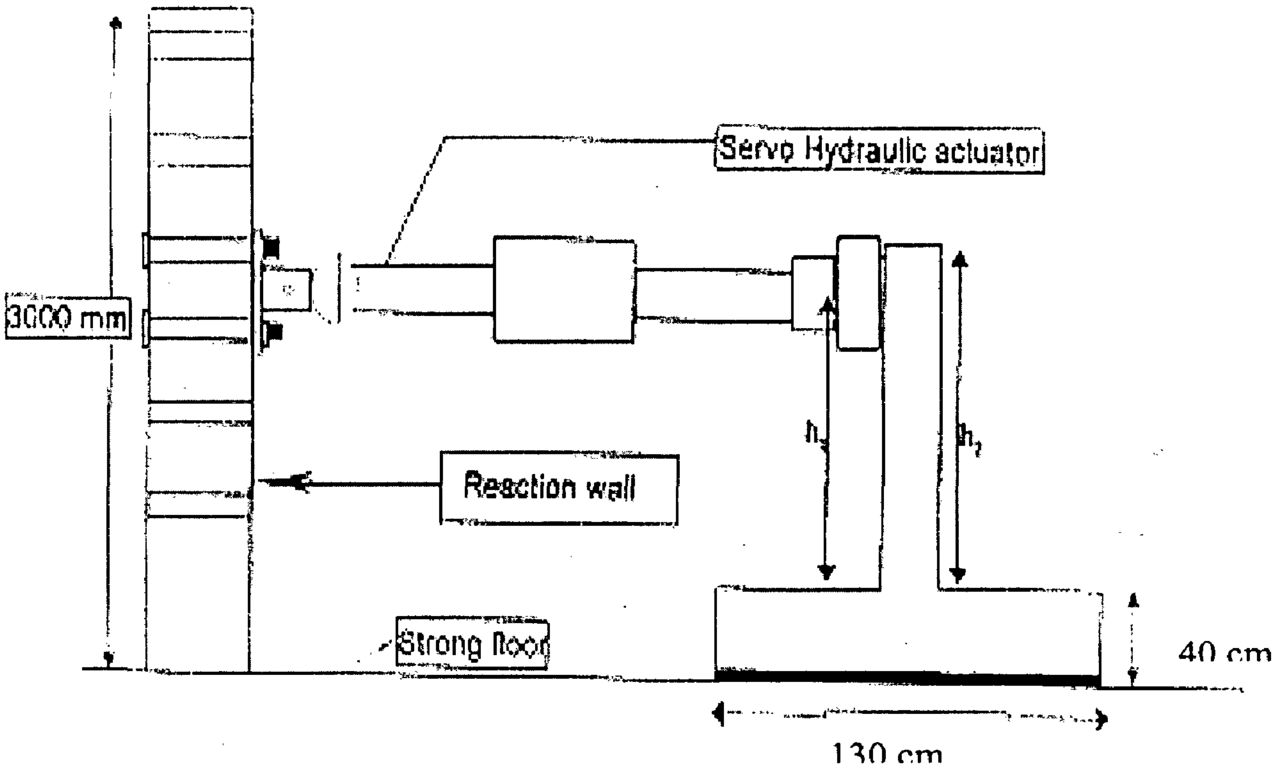


Fig. 3.5 Experimental set up for Quasi-static Test of Bridge Columns

SEISMIC RETROFIT OF BRIDGE COLUMNS

4.1 Introduction

There are two fundamental decisions to be made at the start of RC structure seismic retrofit. The first, based on seismic assessment carried out in the form of experiments, whether the calculated risk of damage or failure warrants retrofit. The second decision will be the level to which the bridge should be retrofitted. After taking decision to retrofit, it can be done in two ways, the first way is to give columns enough ductility and other is to make columns to have enough strength. Considering the cost of project and the strength of foundations the first option is better than second one. However, if the design seismic force is too large to adopt only the first method due to residual deformation or the reasons, we have to adopt both the options for retrofit.

4.2 Seismic Retrofit Methods

A number of retrofit techniques exist for bridges with concrete substructures. These techniques include but are not limited to, steel, composite, and concrete jacketing; wrapping with prestressed strand and reinforcing. For this study two techniques namely steel jacketing and glass fiber reinforced polymer jacketing (GFRP) were considered and explained in next sections.

4.2.1 Steel Jacketing

The use of steel jacketing is the most common retrofit technique for the strengthening of reinforced concrete bridge columns. They are not only the most commonly but also have a recognized design procedure that is generally easy to understand and implement. The design procedures and retrofit method are based on the ability of the jacket, usually oval or circular in shape, to provide sufficient confinement to the column so that a predetermined response level can be achieved. Steel jackets can be used to strengthen a seismically "weak" column, to confine the plastic hinge and lap splice regions of seismically deficient columns, and to enhance the shear strength of columns predisposed to brittle shear failure.

For circular columns two half-shells of steel plate are rolled to a radius of 0.5 to 1.0 in. (12.5 to 25 mm) larger than the column, placed around the column, and site welded up the vertical seams to provide a continuous tube with a smaller annular gap around the column as shown in Figure 4.1. This gap between the column and the jacket is then grouted with a pure cement grout, after flushing with water. Ratios of column diameter to shell thickness in the range 100:1 to 200:1 are common. Typically, a space of about 2 in. (50 mm) is provided between the jacket and any supporting member (footing or cap beam), to avoid the possibility of the jacket acting as compression reinforcement by bearing against the supporting member at large drift angles.

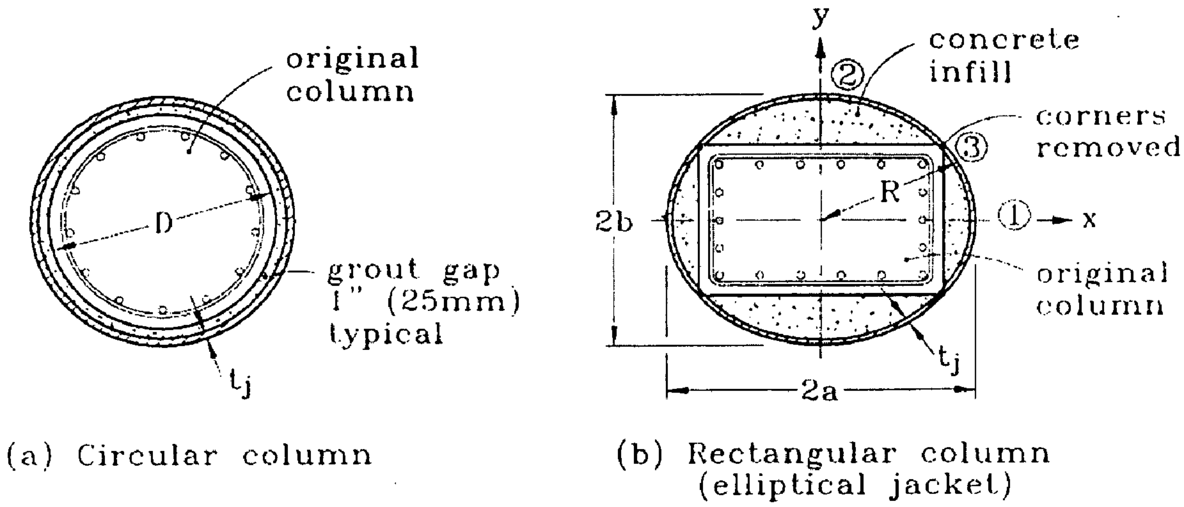


Fig. 4.1 Confinement of columns by steel jacketing

The steel jacket acts as an extremely efficient transverse reinforcement to enhance confinement of plastic hinges and the shear strength of truss shear mechanisms. Rectangular or other noncircular sections have also been retrofitted successfully using steel jackets. In these cases the jacket is rolled to a circular or elliptical shape and the gap between the jacket and column is filled with concrete.

4.2.2 Composite Jacketing

Fiber composite materials generally posses higher strength to weight ratio than conventional materials. Among other advantages of fiber composites are their corrosion resistance, versatility and lightweight. They are made of small fibers such as glass, carbon, Kevlar etc. placed in a resin matrix. The primary load-carrying element within a composite is the fiber. The fiber has a strong influence on the mechanical properties of the composite such as strength and elastic modulus. The resin provides a mechanism for the transfer of load among the fibers. It also protects the fibers from abrasion and the other environmental and chemical attacks. The fibers within the composite can be oriented in a single direction or several directions to optimize the structural performance.

Fiber properties

Glass fibers are the most commonly used composite fiber in the civil engineering dept. The most commonly used glass fiber is an E-glass fiber, which in continuous form have material strengths as follows: density, 2.54 g/cm^3 ; tensile strength 3477 MPa ; elastic modulus, 72.4 GPa ; and the range of diameters, 3-20 μm. The usable value for tensile strength may only be 50% to 75% of these values in a finished product. Fibres in and of themselves have no transverse strength and stiffness properties. It is only when combined with a matrix (binder) material that transverse properties are developed.

Matrix properties

As mentioned previously in the discussion on glass fibers, the fiber only becomes useful as a structural component when combined with a matrix material. The matrix material is in general very compliant and well significant influence the overall strength of the composite, tempering to a large degree, the extremely high-strength values of the fibers alone. Typical values for a polyester resin, a common matrix is presented below. The tensile strength can vary from 34.5 MPa to 103 MPa and tensile modulus from 2 GPa to 4.4 GPa. Another common matrix material, an epoxy resin, has a tensile strength of 55 MPa to 103 MPa and a tensile modulus of 2.75 GPA to 4.10 GPa.

The procedure involves wrapping layers of thin, flexible straps or tapes of resin, impregnated fiber composites around the column in the potential hinge zone or along the entire height of the column if desired as shown in Figure 4.2. An epoxy resin is brushed on the strap while wrapping for interlaminar bonding of the layers of the strap. The composite strap wrapped around the column in this manner, will provide external confinement, prevent crushing and spalling of the concrete shell and prevent buckling of the longitudinal reinforcement. To further enhance the confinement, lateral pressure can be induced on to the column by the use of expansive grout, or by pressure injecting epoxy resin in the gap between the composite strap and the column face. Such gaps can be produced by placing spacers between the composite strap and the column during wrapping operation.

For shear strengthening, the FRP jacket is generally required to cover the entire column height, but for plastic hinge confinement and for lap splice clamping, the FRP jacket is generally only needed in the plastic hinge and nearby regions. Small gaps have been recommended (around 20 mm) between the ends of the FRP jacket and any adjacent transverse structural member to prevent the jacket from direct axial loading (Saadatmanesh et al.1997).

Fig. 4.2 Retrofitting with High-strength Fiber Glass and Epoxy

4.3 Column Retrofit Design Criteria

With poorly confined columns that are expected to sustain large inelastic rotations in plastic hinges, a prime concern will be retrofit design to enhance the ductility capacity. The steps are summarized below:

1. On the basis of the plastic collapse analysis, the required plastic rotation θ_P of the plastic hinge being considered is established.
2. The plastic curvature is found from the expression.

$$\phi_P = \frac{\theta_P}{L_P}$$

where the plastic hinge length is given by $L_P = g + 0.3\ f_y\ d_{bl}$

$g = 0.08\ L$

d_{bl} = diameter of the longitudinal bar.

3. The maximum required curvature is

$$\Phi_m = \Phi_y + \Phi_p$$

Where the equivalent bilinear yield curvature may be found from moment-curvature analysis

4. the maximum required compression strain is given by

$$_{cm} = \Phi_m c$$

where c is the neutral-axis depth(from moment-curvature analysis or flexural strength calculations).

5. The volumetric ratio of confinement required, ρ_s, is given by

$$\rho_s = \Phi_j(\ _{cm})$$

where Φ_j is a materials-dependent relationship between ultimate compression strain and volumetric ratio of jacket confinement.

(i) Steel jacket retrofit: the effective volumetric ratio of confining steel for a circular steel jacket of diameter D is

$$\rho_s = 4\, t_j/D$$

$$_{cm} = 0.004 + \frac{5.6 t_j f_{yj} \varepsilon_{sm}}{D f'_{cc}}$$

where t_j is the jacket thickness, with yield stress f_{yj} and strain at maximum stress of $_{sm}$, and the compression strength of the confined concrete f_{cc}', which is expressed for convenience in Fig in terms of ρ_s and the ratio f_{yj}/f_c', then we have

$$t_j = \frac{0.18(\varepsilon_{cm} - 0.004) D f'_{cc}}{f_{yj} \varepsilon_{sm}}$$

(ii) Composite-material jacket retrofit: Tests on circular columns retrofitted with composite-materials jackets to improve ductility indicate that the confinement effectiveness is more efficient than with steel jackets. It is thought that this is a result of the elastic nature of the jacket material. With steel jacket on unloading, residual plastic strains remain in the jacket, reducing its effectiveness for the next cycle of response, and requiring increased hoop strains for each successive cycle. With materials such as fiberglass and carbon fiber, which have essentially linear stress-strain characteristics up to failure, there is no cumulative damage, and successive cycles to the same displacement result in constant rather than increasing hoop strain.

$$\varepsilon_{cu} = 0.004 + \frac{2.5 \rho_s f_{uj} \varepsilon_{uj}}{f'_{cc}}$$

$$t_j = \frac{0.1(\varepsilon_{cu} - 0.004)Df'_{cc}}{f_{uj}\varepsilon_{uj}}$$

where f_{uj} and ε_{uj} are the ultimate stress and strain of the jacket material.

4.4 Laboratory Testing of Columns

4.4.1 Steel Jacketing

Repair and retrofit of column M1 involved chipping off the cover concrete of the column in the vicinity of the plastic hinge region and wherever cracks are noticeable. After that two half shells of steel plate of thickness 2.5 mm rolled to radius of 15 mm larger than the column radius are positioned over the height of 800 mm and are welded up the vertical seems to provide a continuous tube as shown in Plate 5.5. This gap is grouted with a pure cement grout of 1:3, after flushing with water. A space of about 45mm is provided between the jacket and the footing, to avoid the possibility of the jacket acting as compression reinforcement by bearing against the footing at large drift angles. The plastic hinge length has found to be 310 mm after retrofitting. It was left for 10 days for curing. The retrofitted column was then tested to quasi-static cyclic lateral loading. Plates 5.6 to 5.9 show different stages of retrofitting of model M1 with steel jacketing.

4.4.2 Glass Fiber Reinforced Polymer Jacketing

Repair and retrofit of column M2 involved chipping off the cover concrete of the column in vicinity of cracks through out the height of the column and placing of concrete again. The repaired surface has trowelled and left for curing. After curing the column surface was smoothened by hard rock for removing surface irregularities. The dust has been removed and prepared the column surface for wrapping of GFRP fibers. The GFRP fibers are impregnated with polymer resins and wrapped around the column in a wet lay-up process, with the main fibers oriented in the hoop direction. The GFRP fibers are wrapped to height of 950 mm in 5 continuous layers as shown in Plate 5.14. After that it was left for drying for 7 days. The retrofitted column was then tested to quasi-static cyclic lateral loading.

RESULTS AND DISCUSSION

5.1 General

Quasi-static tests of two reinforced concrete bridge column models (M1 and M2) have been done for evaluating the structural deficiencies and for obtaining load-displacement curves, stiffness degradation, hysteretic loops and ductility. The results obtained from experimental study of both the models M1 and M2 before and after retrofitting are presented below:

5.2 Results of Quasi-Static Tests

5.2.1 Hysteretic Behavior of Model M1 Before Retrofitting

Hysteretic envelope in Figure 5.1 shows the relation between load and displacement. Loading was applied in different number of cycles. Relation between load and displacement remains linear up to ± 13.8 mm, beyond that model has started yielding. Model M1 experienced first flexural cracks at the bottom of the column during the third cycle at 21 kN and increased as the load was increased as shown in Plate 5.1. At the theoretical yield load of 28 kN, initial vertical cracks were noticed on the column. These cracks spread over the mid-height length at displacement ductility (μ_Δ) = 1.1 and the peak lateral load of 32 kN were recorded at this level as shown in Figure 5.2. As testing continued, the lateral capacity progressively decreased, the cover concrete just above the special confining region fell off and the lateral load resistance was reduced to 24 kN as shown in Plate 5.3. The model was tested up to 40 mm stroke (displacement), it has taken maximum of 40 kN load at 30 mm displacement with μ_Δ= 2.12. The maximum loads at different displacements are shown in Table 5.1. The hysteresis loops shown in Figure 5.1 show very significant degradation of stiffness of model M1.

5.2.2 Hysteretic Behavior of Model M1 After Retrofitting with Steel Jacketing

Hysteretic envelop in Figure 5.2 shows the relation between load and displacement of model M1 after steel jacketing. Up to ± 19.35 mm load-displacement curve is linear, beyond this yielding has started. Retrofit Model M1 demonstrated significantly improved performance. Small cracks are visible in critical region at the point μ_Δ= 2.32 load at this level is 48 kN with 45 mm displacement but there is no degradation in load carrying capacity as shown in Plate 5.10. The ideal flexural capacity of the column, of 53 kN was observed at μ_Δ= 2.6 at 50 mm lateral displacement. The maximum loads at different displacements are shown in Table 5.1 and Figure 5.5. The confinement with steel jacketing for a circular column is observed as one of the effective methods of retrofitting for damaged column.

5.2.3 Hysteretic Behavior of Model M2 before Retrofitting

Hysteretic envelop in Figure 5.3 gives the load-displacement relation of model M2 before retrofitting. Up to ± 9.65 mm the curve is linear and after that yielding has started. Model M2 experienced rapid degradation in strength occurred early and with very narrow energy dissipation loops. The lateral strength of Model M2 started to drop quickly at a loading cycle to 30 mm displacement. Small cracks have started at bottom but that are effectively visible at mid-height of the column at load level 28 kN as shown in Plate 5.11. Because of bond failure in mid-height of the column the lateral load carrying capacity was reduced approximately by 80% of the calculated value at the ductility level of μ_Δ= 2.53. The maximum loads at different displacements are given in table 5.2. As lateral displacement increases cracks moved down and complete damage has taken place as spalling of cover concrete as shown in plate 5.13 at load level 26 kN at 50 mm stroke.

5.2.4 Hysteretic Behavior of Model M2 After Retrofitting with GFRP Jacketing

Hysteretic envelop in Figure.5.4 gives the relation of load-displacement of model M2 after retrofitting with GFRP jacketing. Up to ± 15 mm the curve is linear and after that yielding has started. The loading was carried out up to a displacement of 50 mm corresponds to a ratio of lateral displacement to column height of approximately 3.33%. The confinement with GFRP material wrapping increased the load carrying capacity of circular bridge column. It has taken the maximum load of 56 kN at 50 mm displacement with μ_{Δ}= 3.33. Cracks are not visible within the jacketing area but small cracks are visible at top of jacketing at load level 45 kN at 25 mm displacement as shown in plate 5.15 without any degradation of capacity. Cracks got widened and are visible clearly at displacement 50 mm at load level 56 kN as shown in plate 5.16.The maximum loads at different displacement levels are given in Table 5.2 and Figure.5.6.

Table 5.1 Maximum Loads at Different Displacements of Model M1

Before retrofitting		After retrofitting	
Displacement(mm)	**Load(kN)**	**Displacement(mm)**	**Load(kN)**
2	5.977722	5	7.423514
5	13.10369	10	15.13313
7.5	17.8912	15	22.84657
10	21.33593	20	30.88045
12.5	25.24224	25	38.43366
15	28.01556	30	44.3122
17.5	31.67391	35	47.97437
20	33.02052	40	50.05722
30	40.16556	45	51.65561
35	37.39605	50	53.12428
40	24.41825		

Table 5.2 Maximum Loads at Different Displacements of Model M2

Before retrofitting		After retrofitting	
Displacement(mm)	Load(kN)	Displacement(mm)	Load(kN)
2	13.16091	2	4.577706
5	21.84329	5	15.1751
10	28.53819	7.5	23.33105
15	31.50988	10	29.9382
20	32.52079	12.5	34.87068
25	32.37201	14.5	37.96445
30	30.52186	17.5	40.56611
35	28.65644	20	42.97704
40	26.63462	25	45.54055
50	26.79866	30	49.32479
		35	51.81582
		40	53.45998
		45	54.94774
		50	56.83

SUMMARY AND CONCLUSIONS

6.1 Summary

Two circular bridge columns have been constructed and their behavior has been studied under Quasi-static testing facility before and after retrofitting of columns. The models have been constructed according to IS: 456-2000 with special confining reinforcement. The models M1 and M2 are retrofitted with steel jacketing and GFRP jacketing after studying their failure patterns. The Quasi-static test has been carried out to measure the load-displacement behavior of models. The results of an investigation on circular bridge columns before and after retrofitting are presented.

6.2 Conclusions

Based on the results of experimental tests described in this study, the following conclusions are drawn on reinforced concrete circular bridge columns retrofitted with steel and GFRP jacketing.

1) The test showed that before retrofitting of model M1 reached its theoretical flexural strength calculated assuming that longitudinal reinforcement reached its yield strength. The spalling of concrete due to bond degradation of the longitudinal column reinforcement resulted in damage being concentrated in critical region with μ_Δ= 2.12. The measured lateral load Vs lateral displacement hysteresis loops showed very significant stiffness degradation.

2) The model M1 was repaired and retrofitted with steel jacketing in critical damaged region. Retrofitted model has demonstrated response up to 50 mm lateral displacement with concentric crack at retrofitted end. The test also showed that the steel jacketing technique is effective as a repair method for

damaged column by increasing its load carrying capacity. Before retrofitting it has taken 40 kN at 40 mm displacement and after retrofitting its capacity increased to 53 kN at 50 mm displacement with μ_Δ= 2.6.

3) The tests showed that before retrofitting, model M2 reached only about 80% of its theoretical flexural strength calculated. Cracks are observed in the critical region and as testing continued they have spread to mid height of the column. The cover concrete spalled and longitudinal reinforcement is visible with severe damage has been observed at 50 mm displacement with 26 kN load with maximum ductility of μ_Δ= 2.53.

4) Concrete column externally wrapped with the GFRP composite straps showed a significant in both and displacement ductility. The retrofitted model M2 developed very stable load-displacement hysteresis loops up to displacement ductility level of μ_Δ= 3.33, without significant structural deterioration associated with bond failure.

5) It is observed that steel and GFRP jacket retrofitting of circular bridge columns are effective with increase in lateral load carrying capacity of column considerably and significant improvement in ductility.

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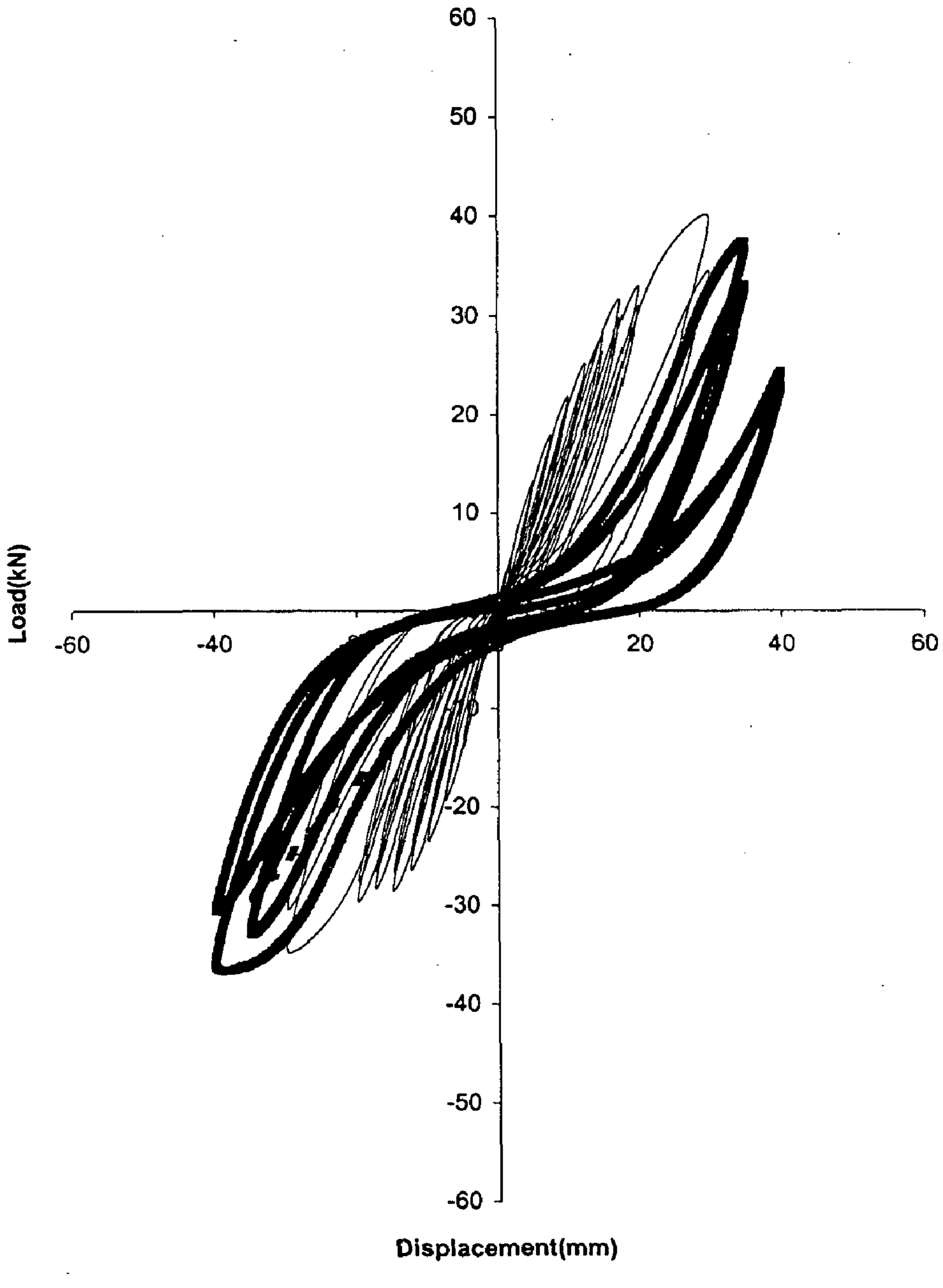


Fig.5.1 Hysteretic Response of as-built Model M1

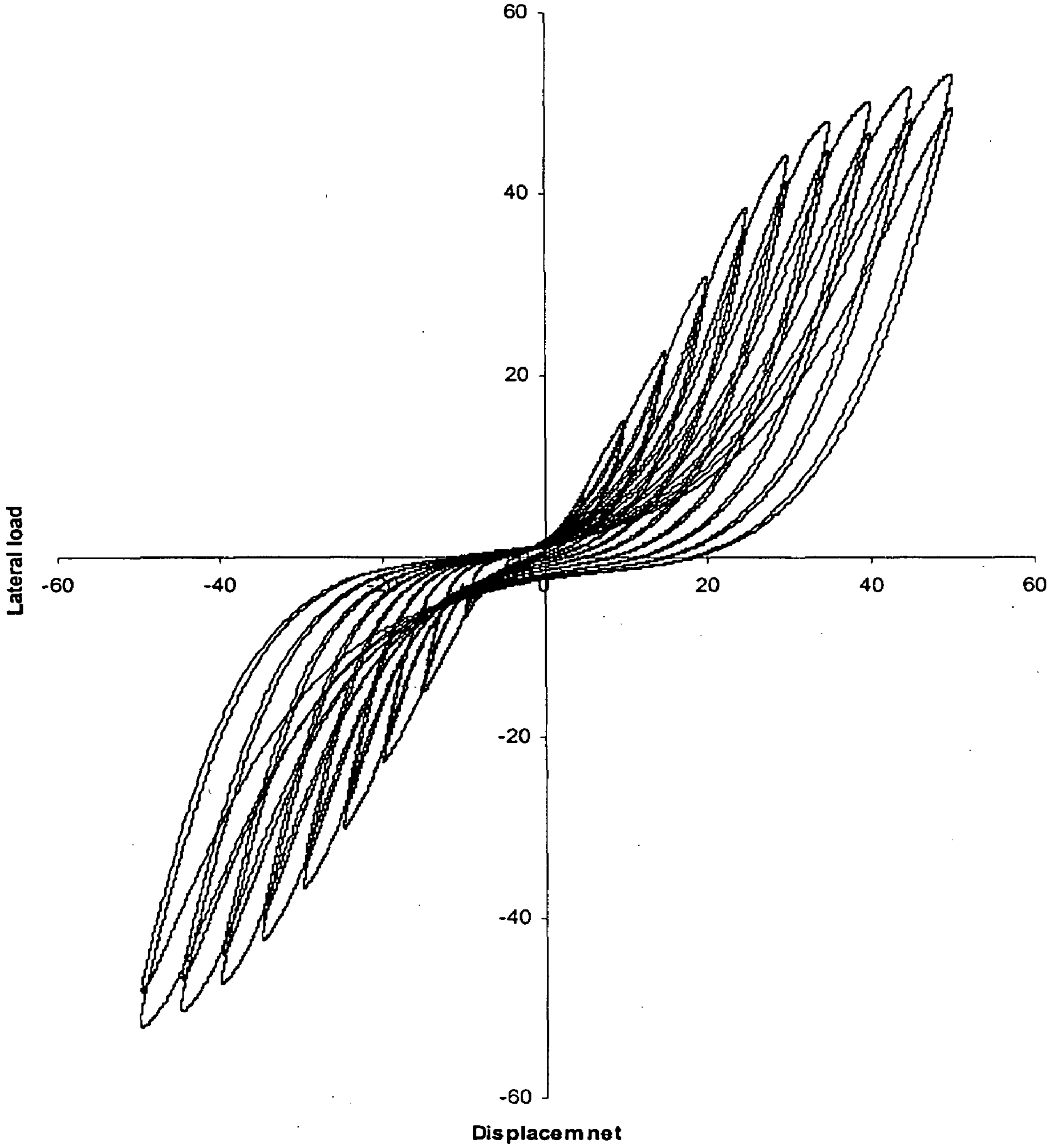


Fig.5.2 Hysteretic Response of Model M1 Retrofitted with steel jacketing

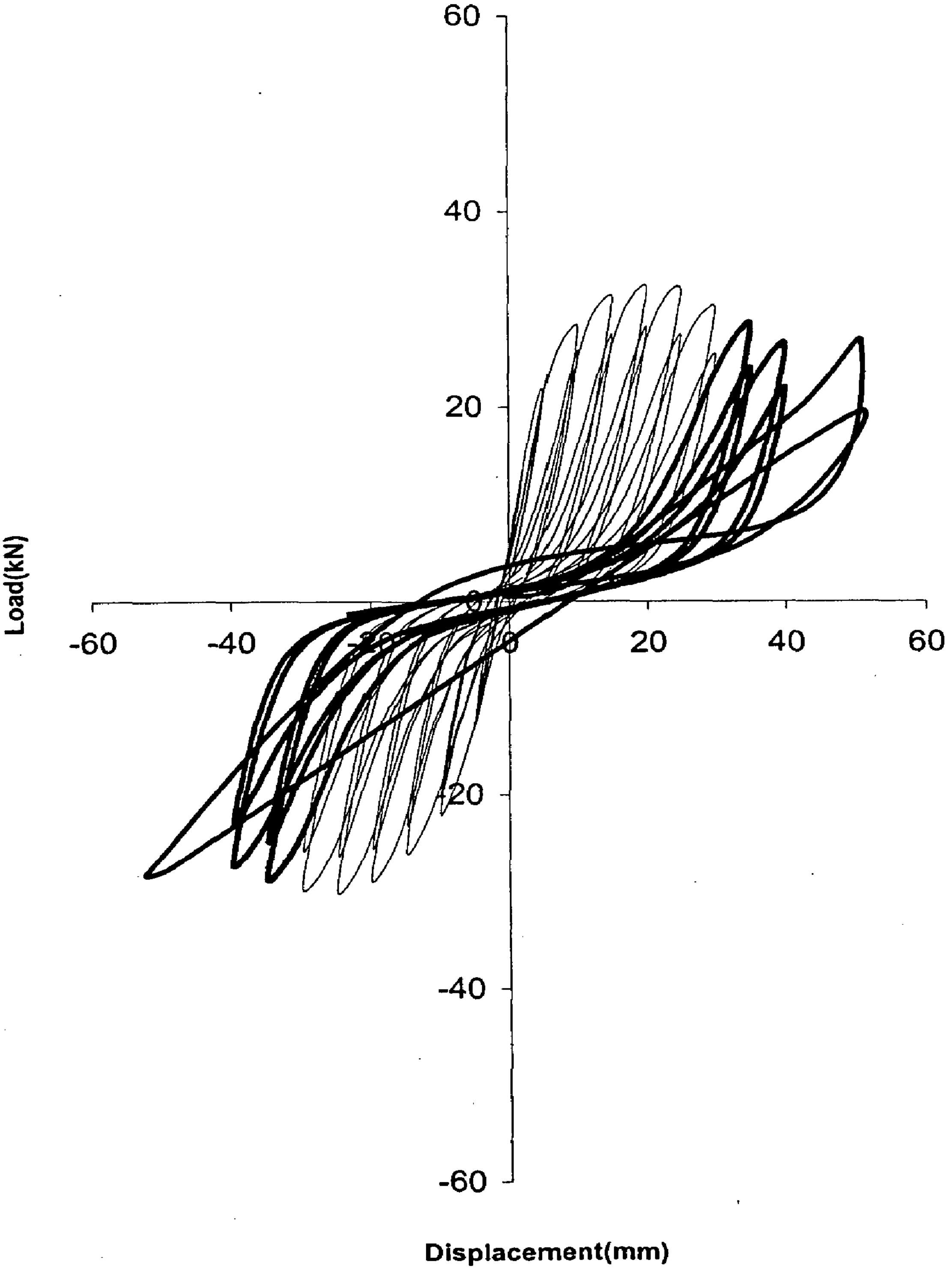


Fig.5.3 Hysteretic Response of as-built Model M2

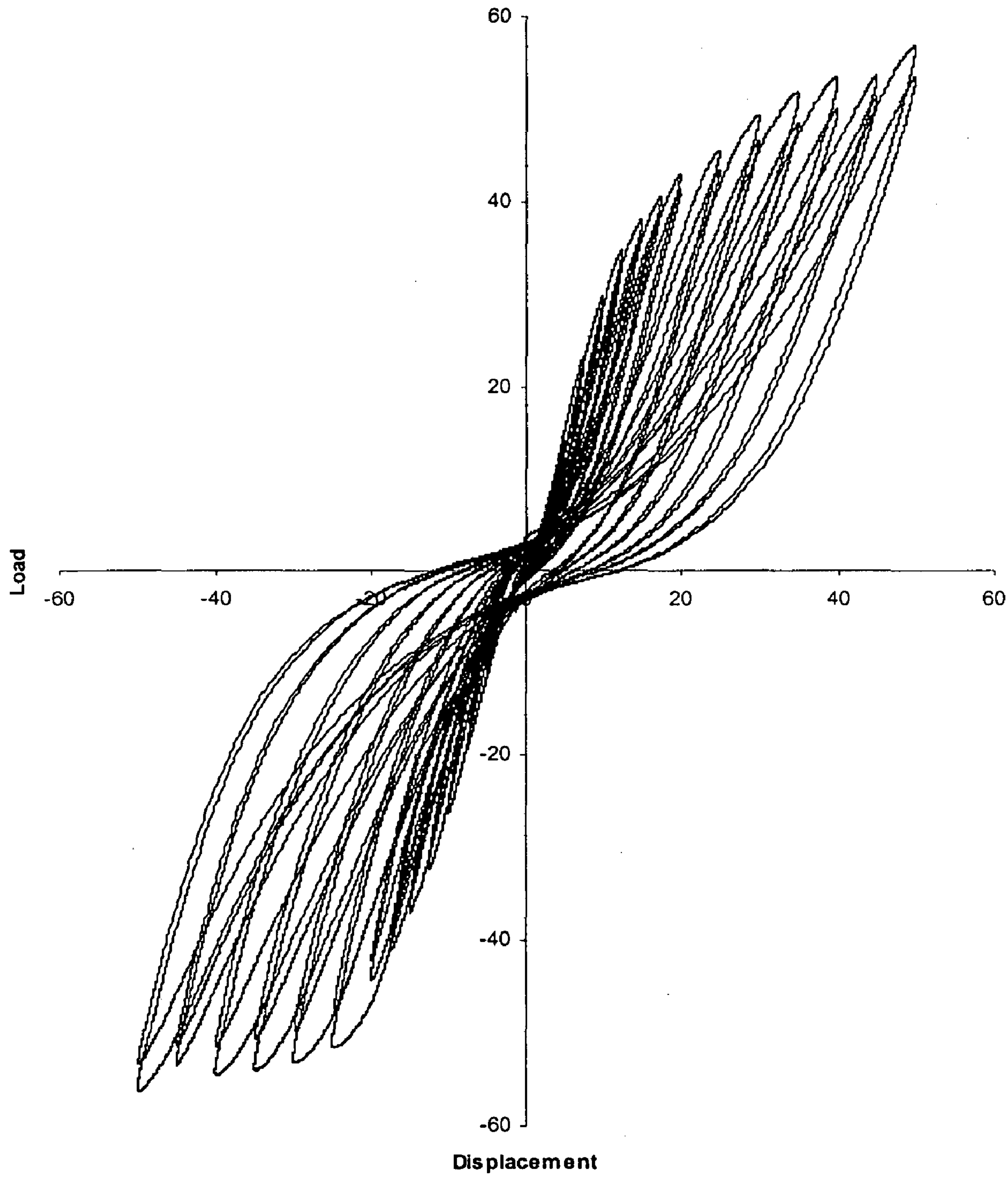


Fig.5.4 Hysteretic Response of Model M2 Retrofitted with GFRP Material

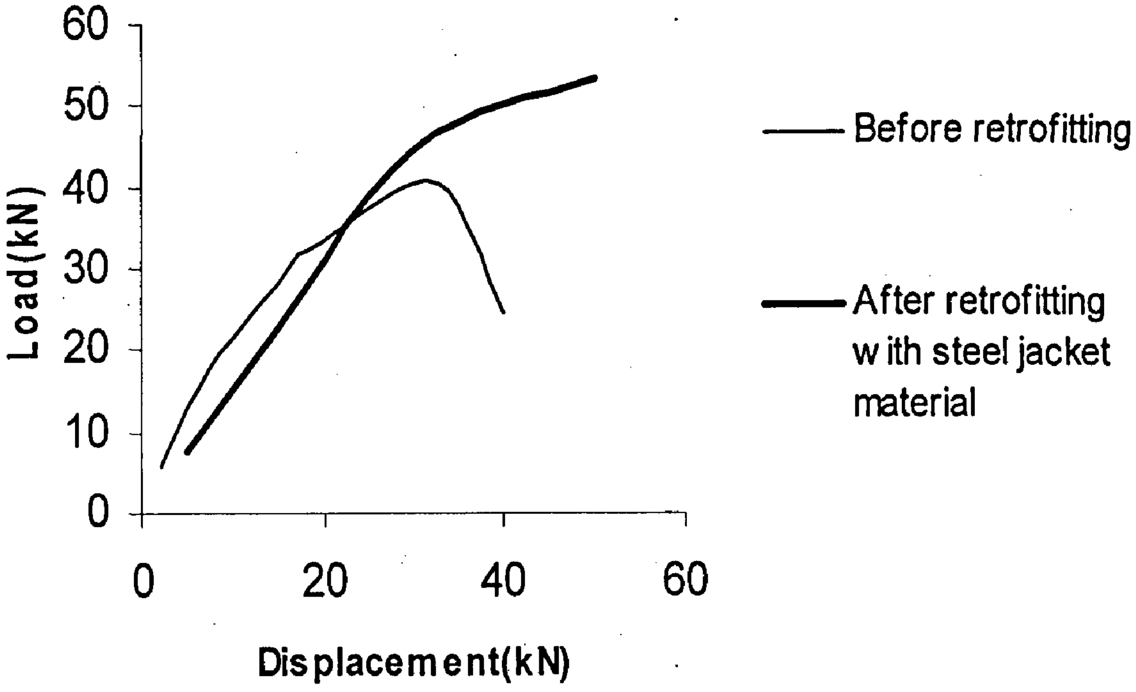


Fig.5.5 Maximum Loads at Different Displacements for Model M1

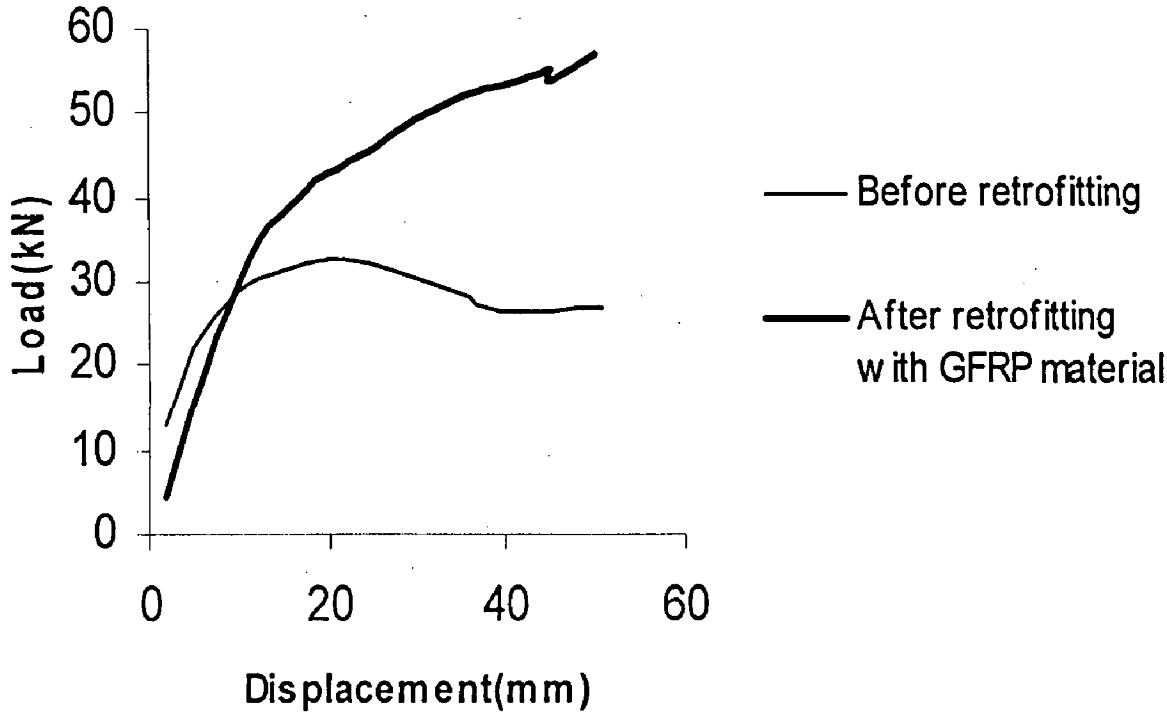


Fig.5.6 Maximum Loads at Different Displacements for Model M2

Plate 3.1 Reinforcement Detailing of Models M1 and M2

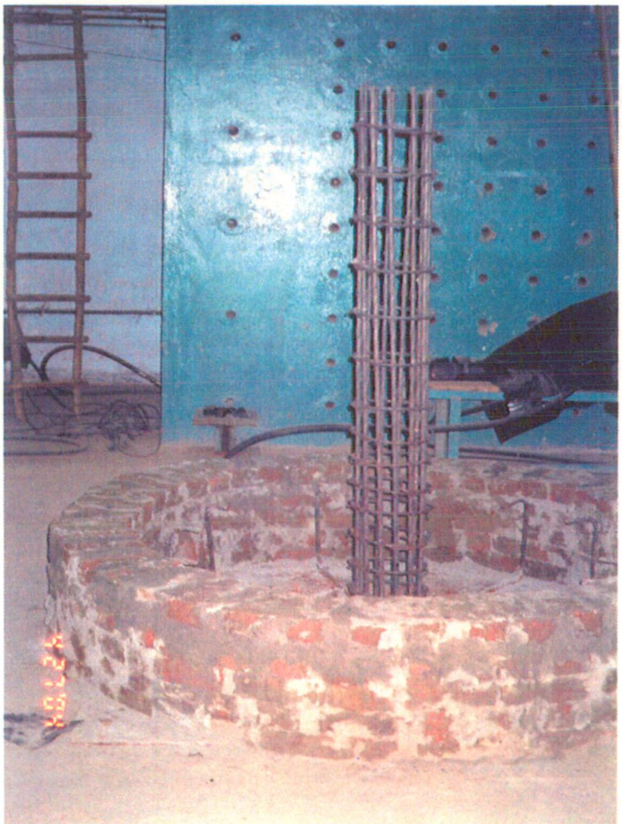


Plate 3.2 Formwork of Model M1 and M2 before Casting

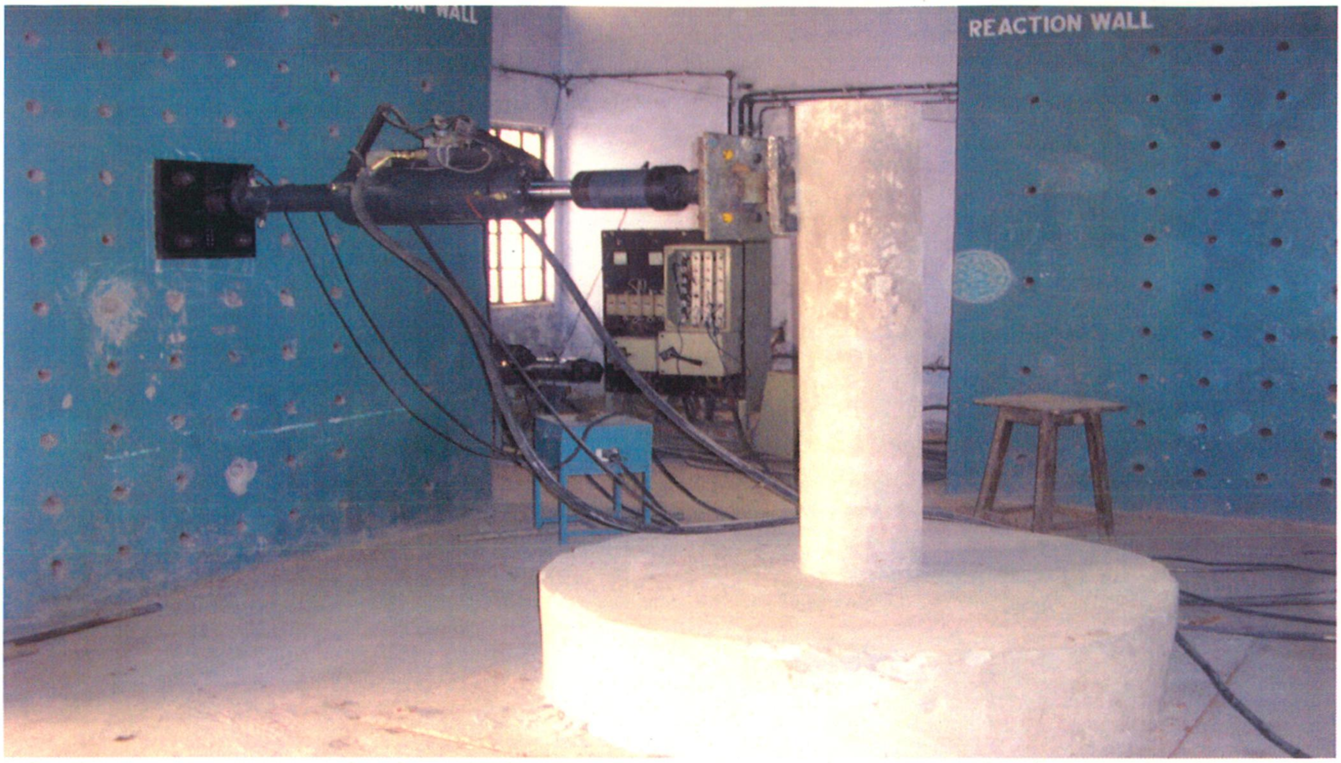


Plate 3.3 Test Setup of Model M1 before Retrofitting

Plate 3.4 Model M2 after Casting

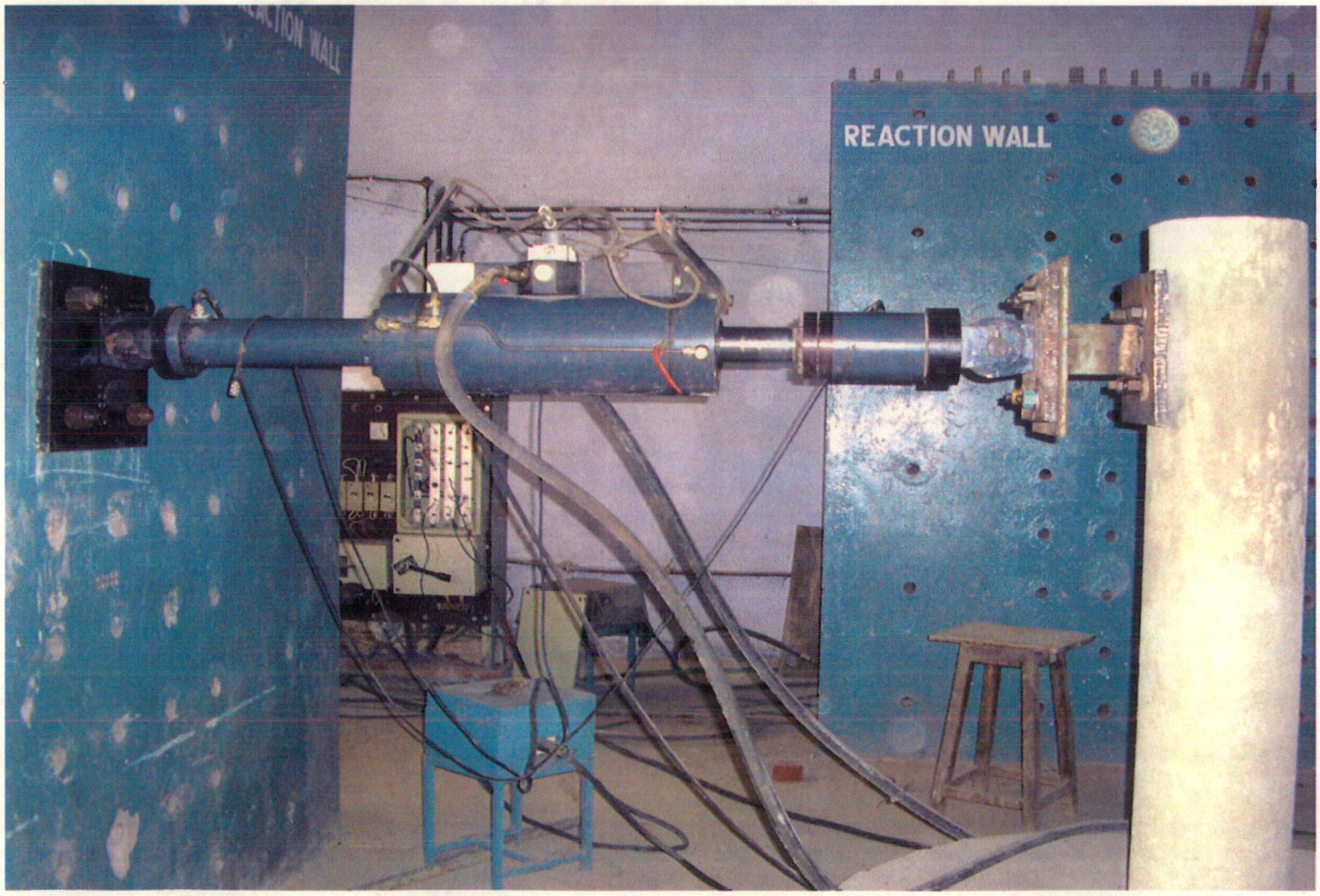
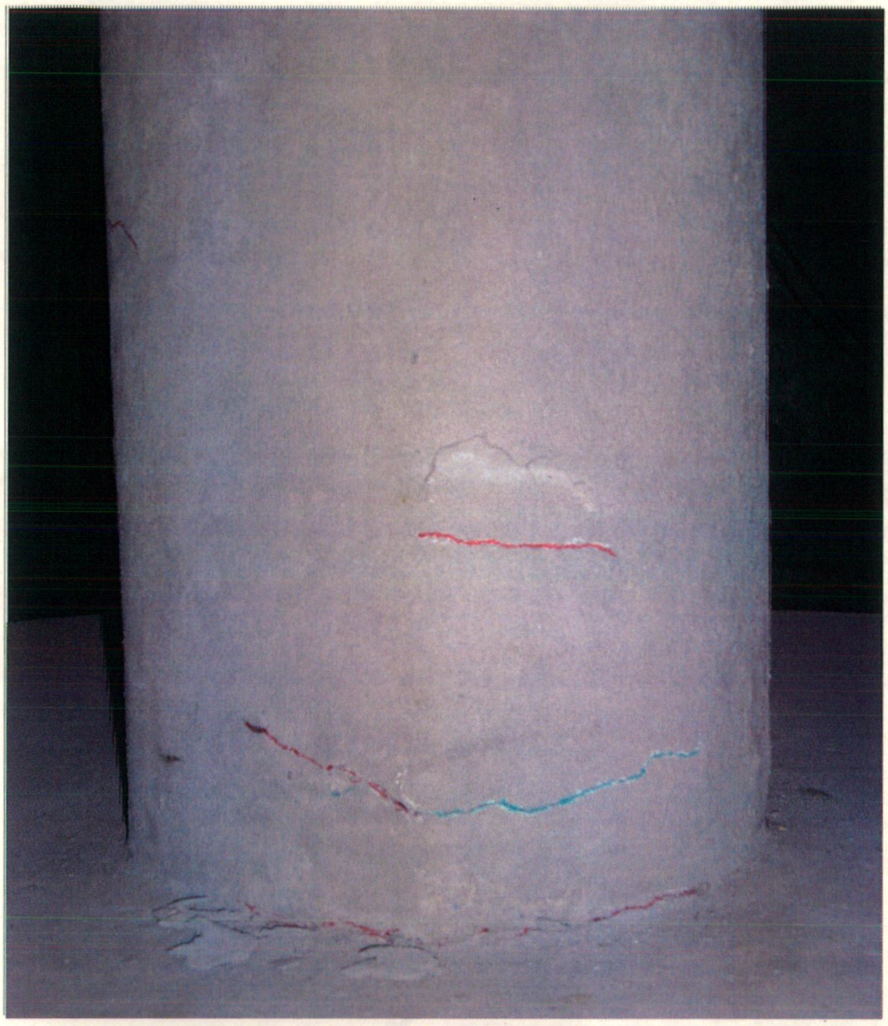


Plate 3.5 Closed-loop Servo Control Hydraulic Actuator

Plate 3.6 Model M1 while Testing (Extreme position of actuator)

Plate 3.7 Test Set-up of Model M2



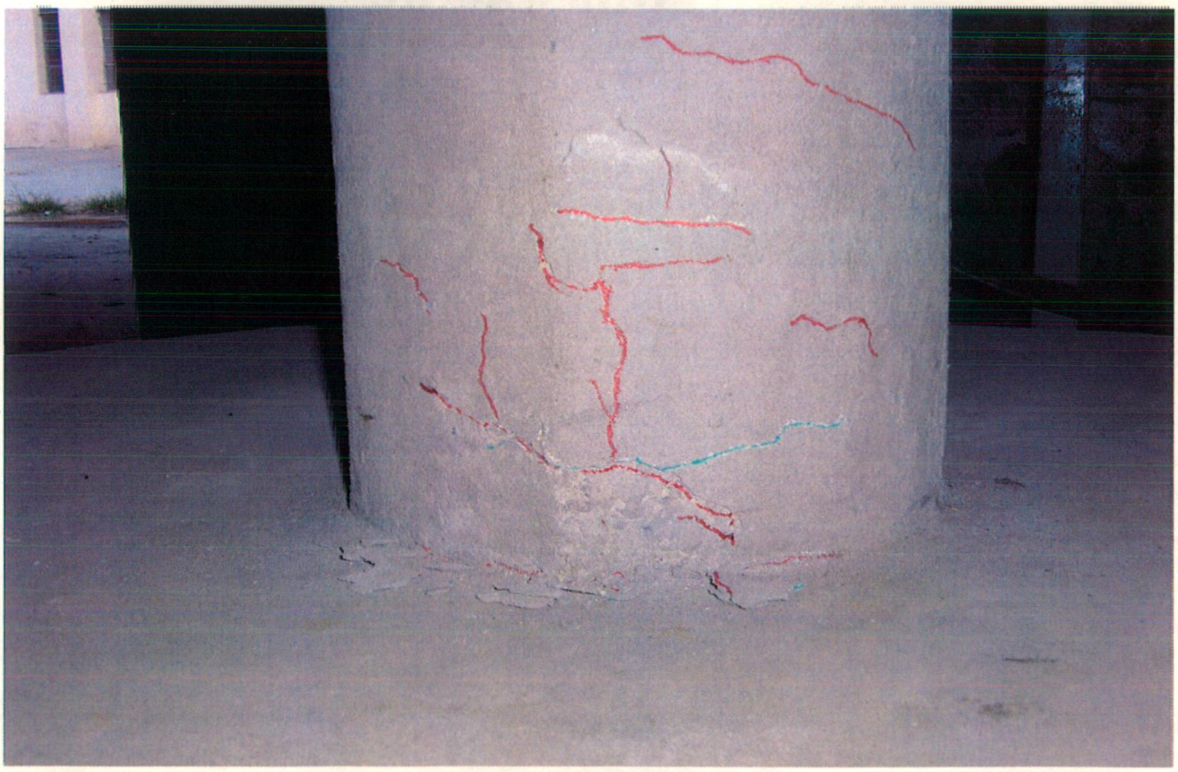


Plate 5.1 Cracks in Model M1

Plate 5.2 Cracks extended to mid height in Model M1

Plate 5.3 Spalling of Concrete in Model M1

Plate 5.4 Model M1 after Testing

Plate 5.5 Model M1 after Retrofitting with steel jacketing

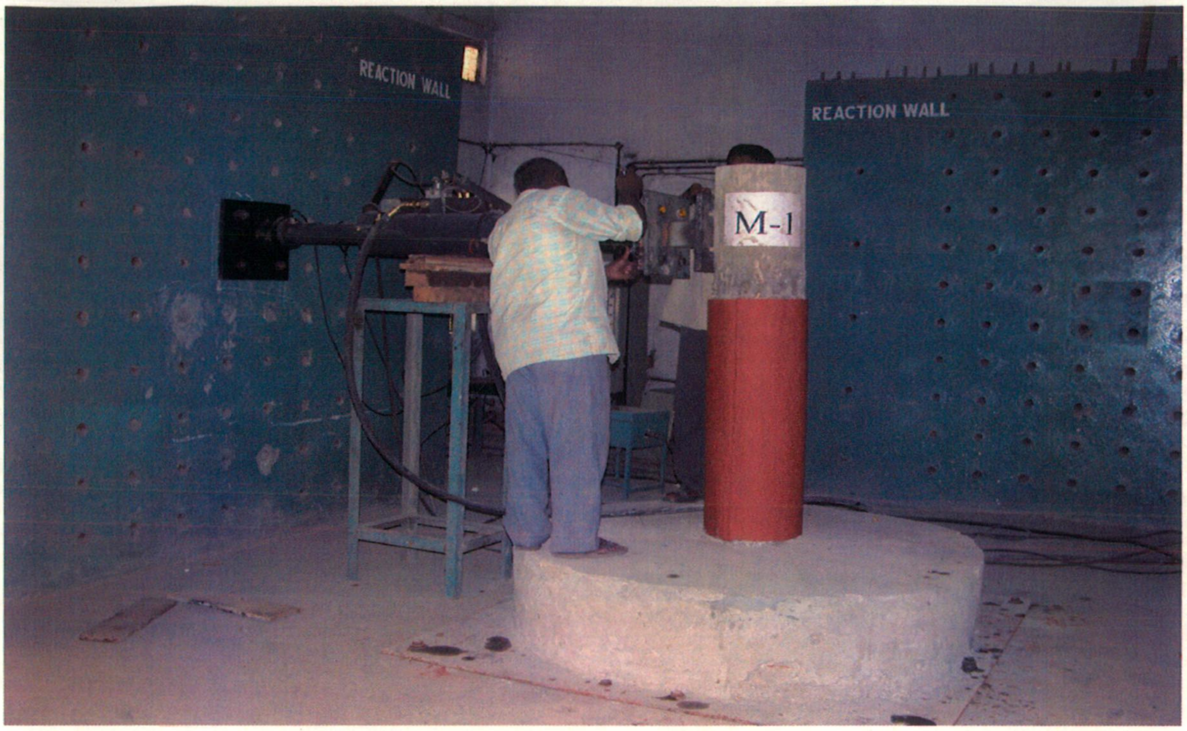


Plate 5.6 Test Set-up of Model M1 after Retrofitting

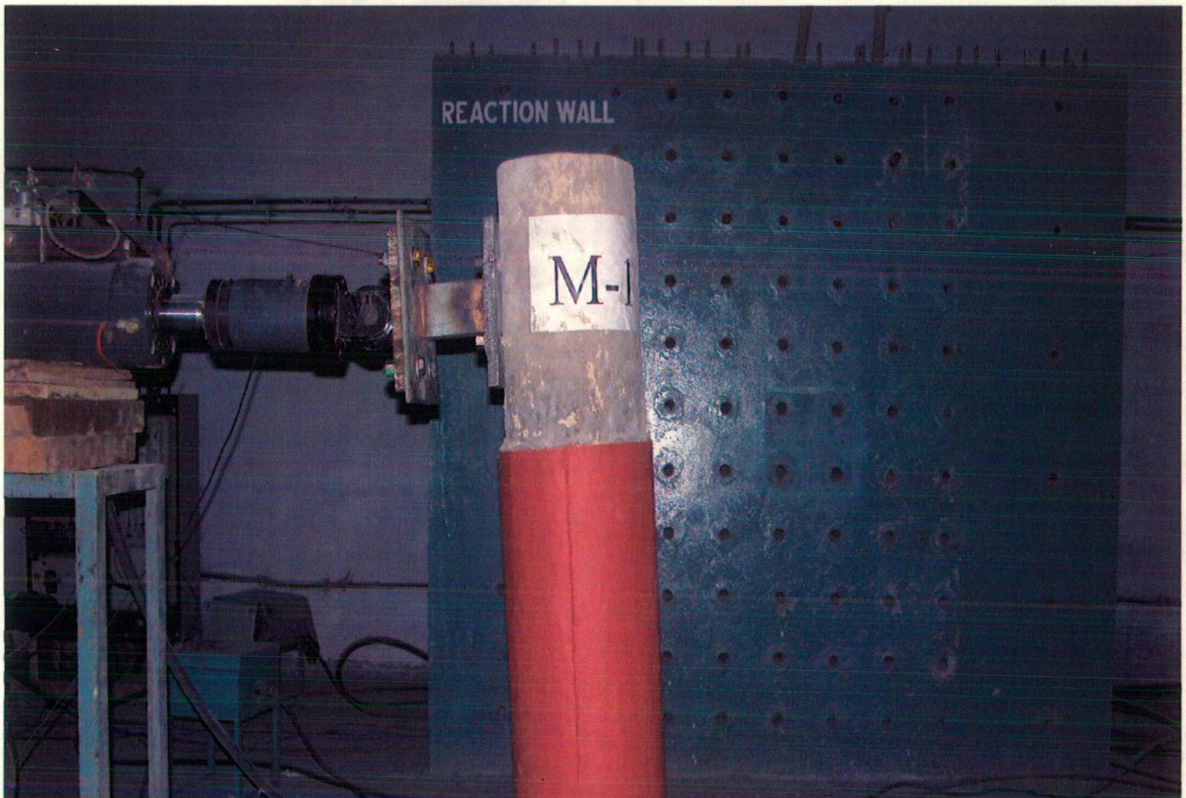


Plate 5.7 Model M1 in Pulling Condition

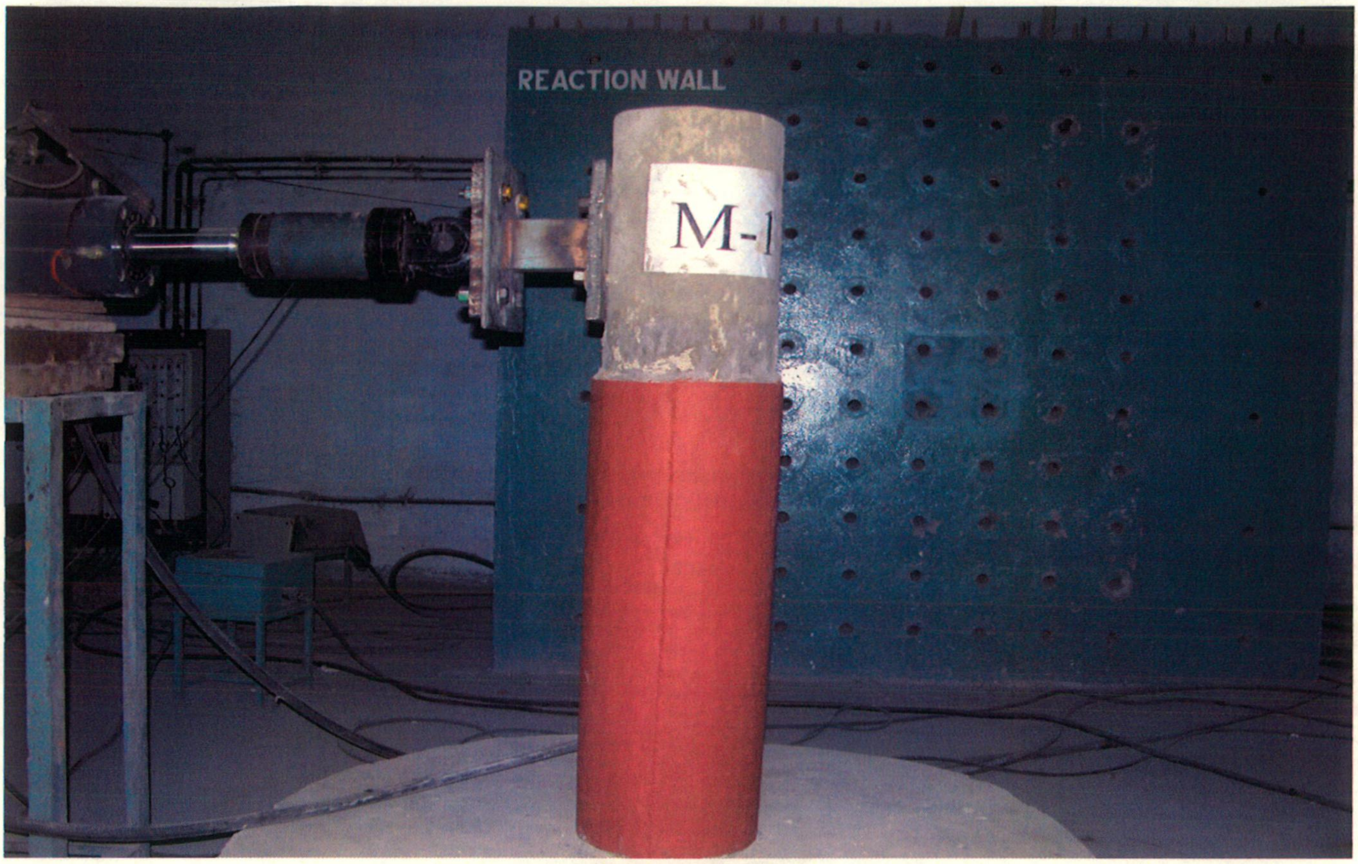


Plate 5.8 Model M1 in pushing condition

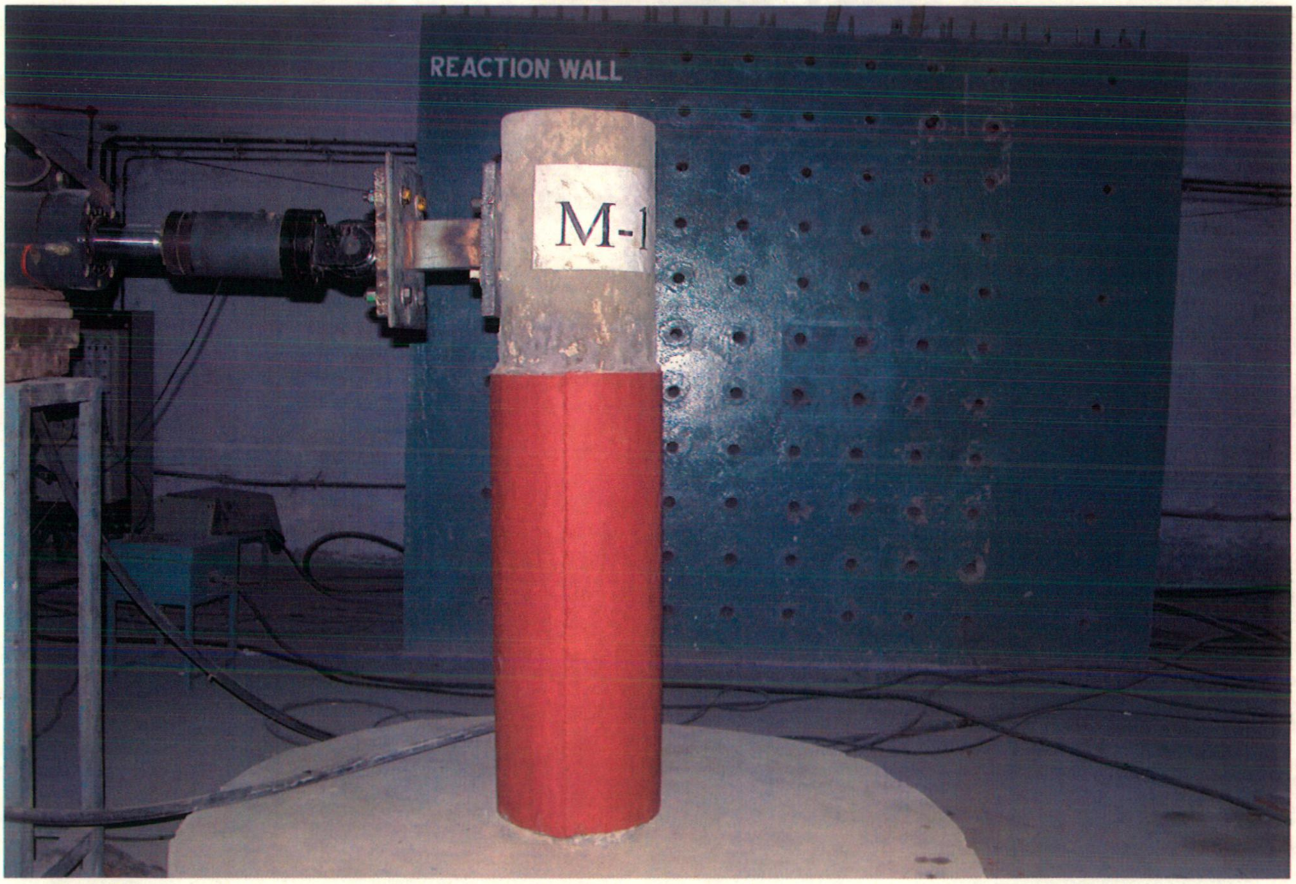


Plate 5.9 Model M1 in zero Position

Plate 5.10 Concentric Cracks in Model M1 after Retrofitting

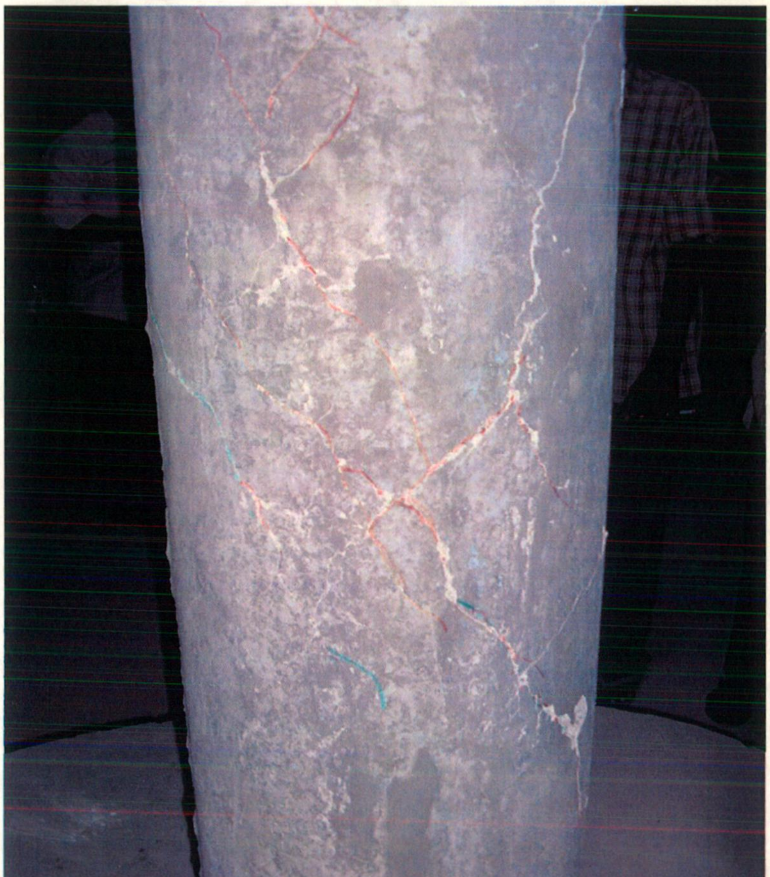
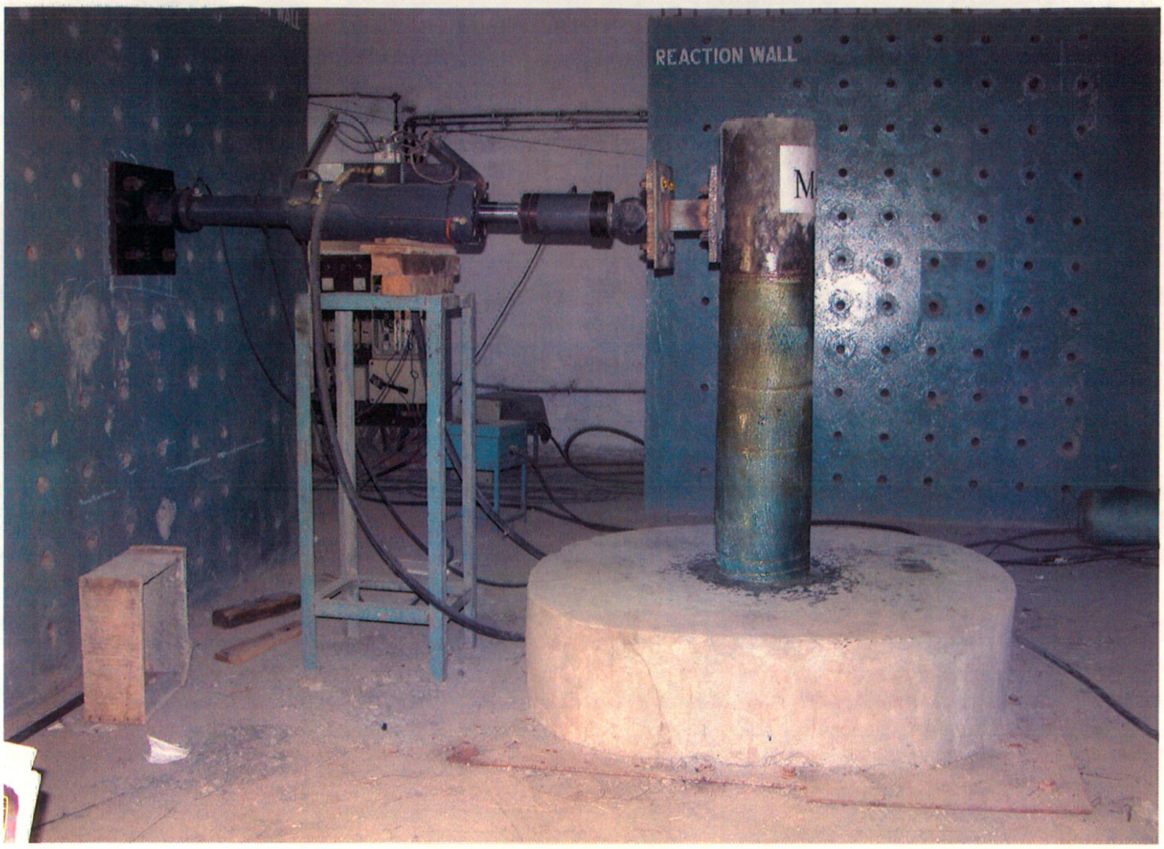


Plate 5.11 Cracks in Model M2

Plate 5.12 Model M2 in Pulling Condition

Plate 5.13 Model M2 after Testing



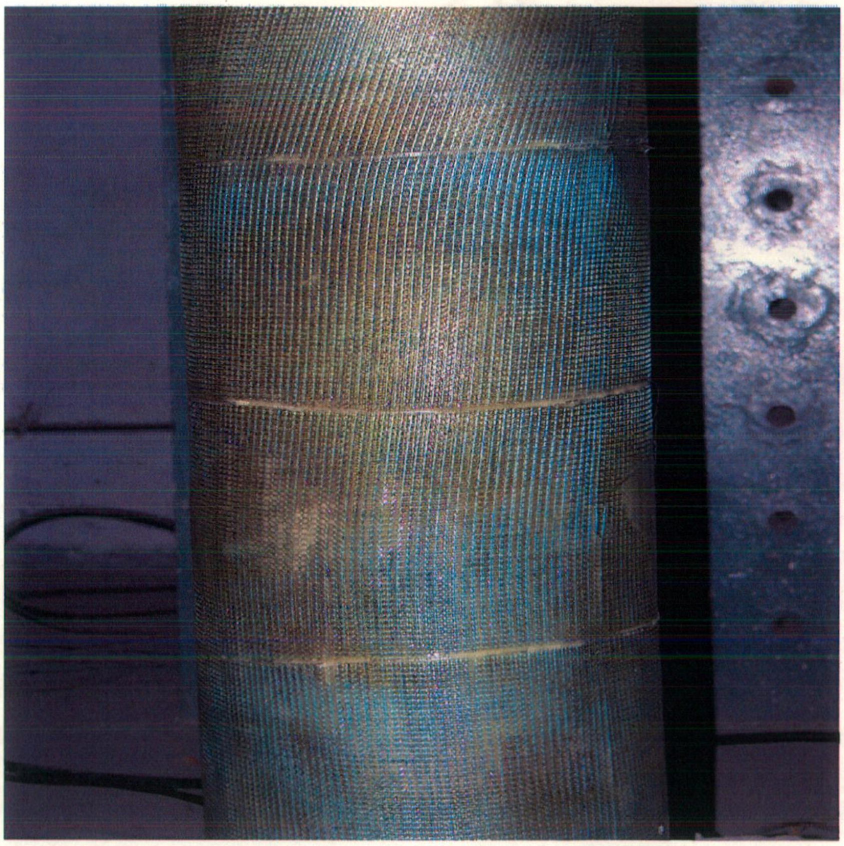


Plate 5.14 Model M2 Retrofitted by GFRP material

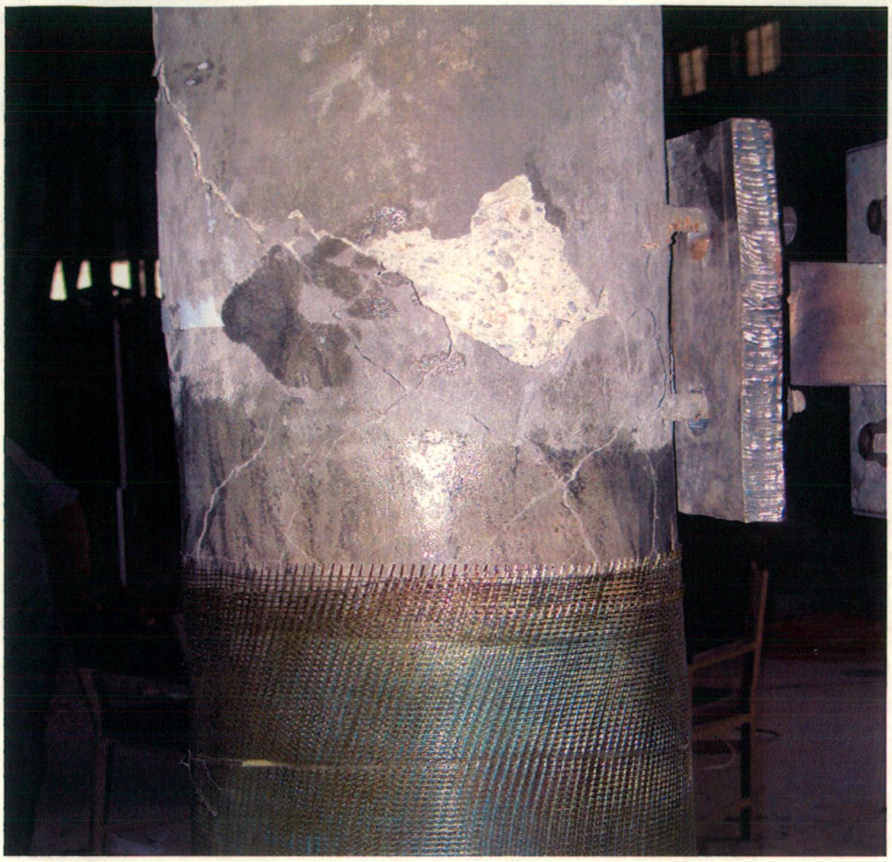


Plate 5.15 Cracks in Model M2

Plate 5.16 Spalling of Concrete at top of the Model M2