

# ***USE OF CARBON FIBER REINFORCED POLYMER SHEETS AS TRANSVERSE REINFORCEMENT IN BRIDGE COLUMNS***

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# ABSTRACT

Performance of bridges during previous earthquakes has demonstrated that many structural failures could be attributed to seismic deficiencies in bridge columns. Lack of transverse reinforcement and inadequate splicing of longitudinal reinforcement in potential plastic hinge regions of columns constitute primary reasons for their poor performance. A number of column retrofit techniques have been developed and tested in the past. These techniques include steel jacketing, reinforced concrete jacketing and use of transverse prestressing (RetroBelt) for concrete confinement, shear strengthening and splice clamping. A new retrofit technique, involving fibre reinforced polymer (FRP) jacketing has emerged as a convenient and structurally sound alternative with improved durability. The new technique, although received acceptance in the construction industry, needs to be fully developed as a viable seismic retrofit methodology, supported by reliable design and construction procedures.

The successful application of externally applied FRP jackets to existing columns, coupled with deteriorating bridge infrastructure, raised the possibility of using FRP reinforcement for new construction. Stay-in-place formwork, in the form of FRP tubes are being researched for its feasibility. The FRP stay-in-place tubes offer ease in construction, convenient formwork, and when left in place, the protection of concrete against environmental effects, including the protection of steel reinforcement against corrosion, while also serving as column transverse reinforcement.

Combined experimental and analytical research was conducted in the current project to i) improve the performance of FRP column jacketing for existing bridge columns, and ii) to develop FRP stay-in-place formwork for new bridge columns. The experimental phase consisted of design, construction and testing of 7 full-scale reinforced concrete bridge columns under simulated seismic loading. The columns represented both existing seismically deficient bridge columns, and new columns in stay-in-place formwork. The

existing columns were deficient in either shear, or flexure, where the flexural deficiencies stemmed from lack of concrete confinement and/or use of inadequately spliced longitudinal reinforcement. The test parameters included cross-sectional shape (circular or square), reinforcement splicing, column shear span for flexure and shear-dominant behaviour, FRP jacket thickness, as well as use of FRP tubes as stay-in-place formwork, with or without internally embedded FRP crossties. The columns were subjected to a constant axial compression and incrementally increasing inelastic deformation reversals.

The results, presented and discussed in this thesis, indicate that the FRP retrofit methodology provides significant confinement to circular and square columns, improving column ductility substantially. The FRP jack also improved diagonal tension capacity of columns, changing brittle shear-dominant column behavior to ductile flexure dominant response. The jackets, when the transverse strains are controlled, are able to improve performance of inadequately spliced circular columns, while remain somewhat ineffective in improving the performance of spliced square columns. FRP stay-in-place formwork provides excellent ductility to circular and square columns in new concrete columns, offering tremendous potential for use in practice.

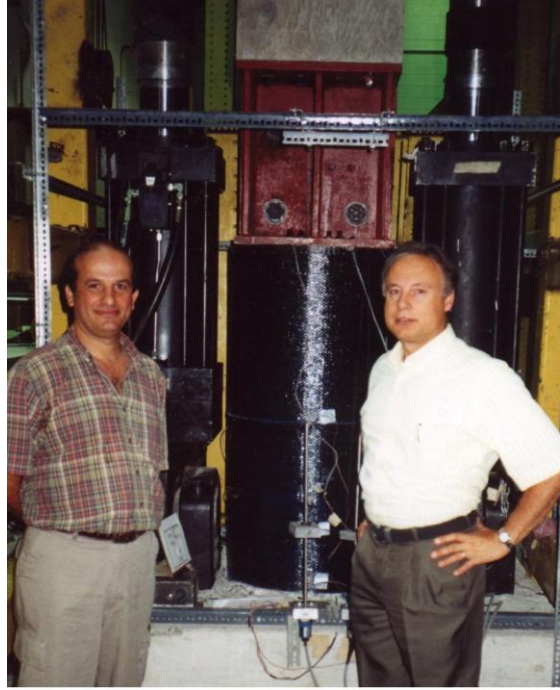
The analytical phase of the project demonstrates that the current analytical techniques for column analysis can be used for columns with external FRP reinforcement, provided that appropriate material models are used for confined concrete, FRP composites and reinforcement steel. Plastic analysis for flexure, starting with sectional moment-curvature analysis and continuing into member analysis incorporating the formation of plastic hinging, provide excellent predictions of inelastic force-deformation envelopes of recorded hysteretic behaviour. A displacement based design procedure adapted to FRP jacketed columns, as well as columns in FRP stay-in-place formwork provide a reliable design procedure for both retrofitting existing columns and designing new FRP reinforced concrete columns.

# FOREWORD

This research work follows one other done by the author. The intention is to introduce innovative retrofitting methods as well as ductility improvement methodologies to deal with seismic related problems and lead to a better understanding the column behaviour during seismic events. The research based on the use of Carbon Fiber Reinforced Polymers (CFRP) as transverse reinforcement for RC bridge columns. Two methods of application are introduced. First one is wrapping seismically deficient existing columns with resin impregnated CFRP fabrics in the potential hinge region and along the entire height of the column to improve the lateral load resistance, a typical inadequacy in pre-1971 structures. Second method includes the use of prefabricated CFRP casing as Stay-in-Place formwork to accommodate fresh concrete mix. These techniques are expected to enhance the ductility and energy dissipation of RC columns, encase them against severe environmental conditions, and protect the reinforcing steel from corrosion and other related problems. With this research, a total of 10 large-scale columns covering different geometries and reinforcement details will have been tested and evaluated. The author acknowledges the respected Yalcin, C., Mes, D., and Beausejour, P. for testing control specimens used within this research for comparison and evaluation. Thanks also go to M. Majeed for helping preparing specimens and for running the MTS.



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# DEFINITIONS AND ACRONYMS

The following definitions clarify terms related to FRP that are not commonly used in the reinforced concrete practice.

|                             |                                                                                                                                                                                                                                                                                                         |
|-----------------------------|---------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>CATALYST:</b>            | A substance that accelerates a chemical reaction.                                                                                                                                                                                                                                                       |
| <b>CFRP:</b>                | Carbon fiber-reinforced polymer.                                                                                                                                                                                                                                                                        |
| <b>COMPOSITE:</b>           | <p>A combination of two or more constituent materials differing in form or composition in macroscale.</p> <p>Note: the constituents retain their identities; that is, they don't dissolve or merge completely into one another, although they act in concert.</p>                                       |
| <b>CONCRETE SUBSTRATE:</b>  | The existing concrete or any cementitious repair material used to repair or replace the existing concrete. The substrate can consist entirely of existing concrete, entirely of repair materials, or of a combination of both. The substrate includes the surface to which the FRP system is installed. |
| <b>CROSS-LINK:</b>          | A chemical bond between polymer molecules.                                                                                                                                                                                                                                                              |
| <b>CURE OF FRP SYSTEMS:</b> | The process of causing the irreversible change in the properties of a thermosetting resin by chemical reaction. Cure is typically accomplished by addition of curing (cross-linking) agents or initiators, with or without heat and pressure.                                                           |
| <b>CURING AGENT:</b>        | A catalytic or reactive agent that causes polymerization when added to resin. Also called hardener or initiator.                                                                                                                                                                                        |

|                         |                                                                                                                                                                            |
|-------------------------|----------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>DEBONDING:</b>       | A separation at the interface between the substrate and the adherent material.                                                                                             |
| <b>DEGRADATION:</b>     | A decline in the quality of the mechanical properties of the material.                                                                                                     |
| <b>DELAMINATION:</b>    | A separation along a plane parallel to the surface, as in separation of the layers of FRP laminates from each other.                                                       |
| <b>DURABILITY, FRP:</b> | The ability of a material to resist weathering action, chemical attack, abrasion, and other conditions of service.                                                         |
| <b>EPOXY:</b>           | A thermosetting polymer that is the reaction product of epoxy resin and amino hardener.                                                                                    |
| <b>EPOXY RESIN:</b>     | A class of organic chemical-bonding systems used in the preparation of a special coating or adhesives for concrete.                                                        |
| <b>FABRIC:</b>          | Arrangement of fibers held together in two dimensions. A fabric can be woven, nonwoven, knitted, or stitched.                                                              |
| <b>FIBER:</b>           | Any fine thread-like natural or synthetic object of mineral or organic origin. This term is generally used for materials whose length is at least 100 times its diameter.  |
| <b>FIBER, ARAMID:</b>   | Highly oriented organic fiber derived from polyamide incorporating into an aromatic ring structure.                                                                        |
| <b>FIBER, CARBON:</b>   | Fiber produced by heating organic precursor materials containing a substantial amount of carbon, such as rayon, polyacrylonitrile (PAN), or pitch in an inert environment. |

|                                  |                                                                                                                                                                                                                                                                                           |
|----------------------------------|-------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>FIBER, GLASS:</b>             | Fiber drawn from an inorganic product of fusion that has cooled without crystallizing. Types of glass fibers include alkali resistant (AR-glass), general purpose (E-glass), and high strength (S-glass).                                                                                 |
| <b>FIBERGLASS:</b>               | A composite material consisting of glass fibers in resin.                                                                                                                                                                                                                                 |
| <b>FIBER REINFORCED POLYMER:</b> | A general term for a composite material that consists of a polymer matrix reinforced with cloth, mat, strands, or any other fiber form. See <b>COMPOSITE</b> .                                                                                                                            |
| <b>FRP:</b>                      | Fiber reinforced polymers; formerly, fiber-reinforced plastic.                                                                                                                                                                                                                            |
| <b>GRID, FRP:</b>                | A two-dimensional (planar) or three-dimensional (spatial) rigid array of interconnected FRP bars that form a contiguous lattice that can be used to reinforce concrete. The lattice can be manufactured with integrally connected bars or made of mechanically connected individual bars. |
| <b>HARDENER, FRP:</b>            | In a two-component adhesive or coating, the chemical component that causes the resin component to cure.                                                                                                                                                                                   |
| <b>IMPREGNATE, FRP:</b>          | To saturate the fibers with resin.                                                                                                                                                                                                                                                        |
| <b>INITIATOR:</b>                | A source of free radicals, which are groups of atoms that have at least one unpaired electron, used to start the curing process for unsaturated polyesters and vinyl ester resin. Peroxides are the most common source of free radicals. See <b>CATALYST</b> .                            |
| <b>INTERFACE:</b>                | The boundary or surface between two different, physically distinguishable media. On fibers, the contact area between fibers and coating.                                                                                                                                                  |

|                    |                                                                                                                                                                                                                                                                                    |
|--------------------|------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------------|
| <b>LAMINATE:</b>   | One or more layers of fiber bound together in a cured resin matrix.                                                                                                                                                                                                                |
| <b>LAYUP:</b>      | The process of placing the FRP reinforcing material in a position for molding.                                                                                                                                                                                                     |
| <b>MATRIX:</b>     | In the case of fiber-reinforced polymers, the materials that serve to bind the fibers together, transfer load to the fibers, and protect them against environmental attack and damage due to handling.                                                                             |
| <b>PITCH:</b>      | Petroleum or coal tar precursor base used to make carbon fiber.                                                                                                                                                                                                                    |
| <b>PLY:</b>        | A single layer of fabric; multiple plies, when molded together, make up the laminate.                                                                                                                                                                                              |
| <b>POLYMER:</b>    | A high molecular weight organic compound (natural or synthetic) containing repeating units.                                                                                                                                                                                        |
| <b>PULTRUSION:</b> | A continuous process for manufacturing composites that have a uniform cross-sectional shape. The process consists of pulling a fiber-reinforcing material through a resin impregnation bath then through a shaping die where the resin is subsequently cured.                      |
| <b>RESIN:</b>      | Polymeric material that is rigid or semirigid at room temperature, usually with a melting point above room temperature.                                                                                                                                                            |
| <b>SHEET, FRP:</b> | A dry, flexible ply used in wet layup FRP systems. Unidirectional FRP sheets consist of continuous fibers aligned in one direction and held together in-plane to create a ply of finite width and length. Fabrics are also referred to as sheets. See <b>FABRIC</b> , <b>PLY</b> . |

**WET LAYUP:**

A method of making a laminate product by applying the resin system as a liquid when the fabric is put in place.

**WET-OUT:**

The process of coating or impregnating fabric in which all voids between the strands are filled with resin.



# NOTATIONS

|                 |                                                                                                                     |
|-----------------|---------------------------------------------------------------------------------------------------------------------|
| $A_c$           | cross-sectional area of the core of a compression member measured to the centreline of the perimeter hoop or spiral |
| $A_g$           | gross area of concrete section                                                                                      |
| $A_{Fh}$        | total area of rectangular FRP hoop reinforcement in each cross-sectional direction                                  |
| $A_{st}$        | total area of longitudinal steel reinforcement, mm <sup>2</sup>                                                     |
| $A_v$           | area of shear reinforcement perpendicular to the axis of a member within a distance $s$                             |
| $b_c$           | core dimension measured center to center of perimeter hoop                                                          |
| $C_j$           | modification factor for longitudinal reinforcement details                                                          |
| $d_b$           | longitudinal bar diameter                                                                                           |
| $D_c$           | Column diameter or outside dimension of the column                                                                  |
| $E$             | modulus of elasticity                                                                                               |
| $f'_c$          | cylinder compressive strength of concrete specimens                                                                 |
| $f_u$           | ultimate strength of reinforcing steel                                                                              |
| $f_y$           | yield strength of reinforcing steel                                                                                 |
| $F$             | applied lateral load                                                                                                |
| $L$             | shear span (effective height of the column)                                                                         |
| $M$             | applied moment                                                                                                      |
| $M_y$           | yielding moment                                                                                                     |
| $t_j$           | CFRP jacket thickness                                                                                               |
| $\Delta$        | lateral tip displacement of the column                                                                              |
| $\Delta_y$      | first yield displacement                                                                                            |
| $\varepsilon_h$ | strain of reinforcing steel at strain hardening                                                                     |
| $\varepsilon_u$ | ultimate strain of reinforcing steel                                                                                |
| $\varepsilon_y$ | yield strain of reinforcing steel                                                                                   |
| $\theta_h$      | total rotation at $h$ from the base of the column                                                                   |
| $\theta_{h/2}$  | total rotation at $h/2$ from the base of the column                                                                 |
| $\theta_0$      | anchorage slip rotation at the base of the column                                                                   |
| $\rho_l$        | longitudinal reinforcement ratio                                                                                    |

|          |                                                                                                                                                        |
|----------|--------------------------------------------------------------------------------------------------------------------------------------------------------|
| $\mu$    | displacement ductility ratio ( $\Delta/\Delta_y$ )                                                                                                     |
| $d$      | distance from extreme compression fibre to centroid of tension reinforcement                                                                           |
| $D$      | diameter of circular columns or dimension in the loading direction of rectangular columns                                                              |
| $E_c$    | modulus of elasticity of concrete                                                                                                                      |
| $E_F$    | elastic modulus of FRP composite                                                                                                                       |
| $f_F$    | stress in FRP composite                                                                                                                                |
| $f_{Fd}$ | design stress level in the FRP wrap                                                                                                                    |
| $f_{Fh}$ | design stress level in FRP transverse confinement reinforcement                                                                                        |
| $f_{Fu}$ | ultimate tensile strength of FRP composites                                                                                                            |
| $f_{le}$ | equivalent lateral pressure that produces the same effect as uniformly applied pressure                                                                |
| $f_y$    | specified yield strength of steel reinforcement                                                                                                        |
| $f'_c$   | specified compressive strength of concrete                                                                                                             |
| $h_c$    | cross-sectional dimension of column core                                                                                                               |
| $I_g$    | moment of inertia of gross concrete section about the centroidal axis, neglecting the reinforcement                                                    |
| $k$      | strength enhancement coefficient                                                                                                                       |
| $k_1$    | coefficient that relates confinement pressure to strength enhancement                                                                                  |
| $k_2$    | coefficient that reflects efficiency of confinement reinforcement                                                                                      |
| $k_3$    | coefficient to reflect effect of concrete strength                                                                                                     |
| $k_4$    | coefficient to reflect effect of transverse steel strength                                                                                             |
| $k_c$    | confinement efficiency coefficient                                                                                                                     |
| $\ell_o$ | minimum length measured from the face of the joint along the axis of the column, over which FRP reinforcement must be provided for seismic confinement |
| $P$      | specified axial load on column section                                                                                                                 |
| $P_f$    | factored axial load                                                                                                                                    |
| $P_o$    | nominal unconfined axial load capacity of column, taken as $\alpha_1 f'_c (A_g - A_{st}) + f_y A_{st}$ ,                                               |
| $P_{ro}$ | factored axial load resistance at zero eccentricity                                                                                                    |
| $s$      | spacing of transverse reinforcement or the spiral pitch                                                                                                |

|                    |                                                                                                  |
|--------------------|--------------------------------------------------------------------------------------------------|
| $s_l$              | spacing of longitudinal reinforcement, laterally supported by corner of hoop or hook crosstie    |
| $f_{st}$           | tensile stress in transverse reinforcement at peak concrete stress                               |
| $t_j$              | thickness of the FRP jacket                                                                      |
| $V_c$              | shear resistance provided by concrete                                                            |
| $V_F$              | shear resistance provided by FRP jacket                                                          |
| $V_r$              | factored shear resistance                                                                        |
| $V_s$              | factored shear resistance provided by the transverse steel reinforcement                         |
| $Y_p$              | parameter to account for the effect of axial load on seismic confinement requirement             |
| $\beta_c$          | confinement efficiency parameter for circular columns                                            |
| $\delta$           | design lateral drift ratio (i.e., horizontal drift/building height)                              |
| $\lambda$          | factor to account for concrete density                                                           |
| $\varepsilon_{cu}$ | ultimate compressive strain of concrete                                                          |
| $\rho_c$           | total transverse steel area in two orthogonal directions divided by corresponding concrete area, |

# CHAPTER 1

## INTRODUCTION

### 1.1 GENERAL

Structures are primarily designed to resist vertical forces that result from gravity loads, with due considerations given to wind and seismic loads. Typical structural elements include foundations, load-bearing walls, beams or girders, columns or bridge piers, slabs or bridge decks. Slabs and bridge decks transfer vertical loads to the supporting beams and girders, which are then carried to columns, bearing walls, or bridge piers, which transfer forces down to the foundation and the supporting soils. This process of transfer of loads follows a load path. Distress in any element along the load path can lead to structural damage or collapse.

Seismic forces are different than gravity and wind loads which act externally on structural elements. Earthquake induced ground accelerations at supports generate accelerations of the mass, generating inertia forces at slab deck/floor locations where the mass tends to concentrate. Horizontal and vertical earthquake forces travel in different load paths, and may result in flexural tension and compression, diagonal tension and compression (due to shear forces), as well as axial and torsional stresses.

While it is possible to design structures that can sustain no damage during an earthquake, such measures are uneconomical for rare and strong earthquakes. Therefore, modern seismic codes aim to reduce seismic risk, while accepting some level of structural damage. In earthquake resistant design of reinforced concrete, structures are designed with lateral-force-resisting systems to withstand the effects of earthquake forces. These seismic systems often make the structure stronger as well as stiffer against horizontal forces, thus minimizing the amount of relative lateral movement and consequential damage. Seismic systems are usually designed to resist forces that result from horizontal ground excitations.

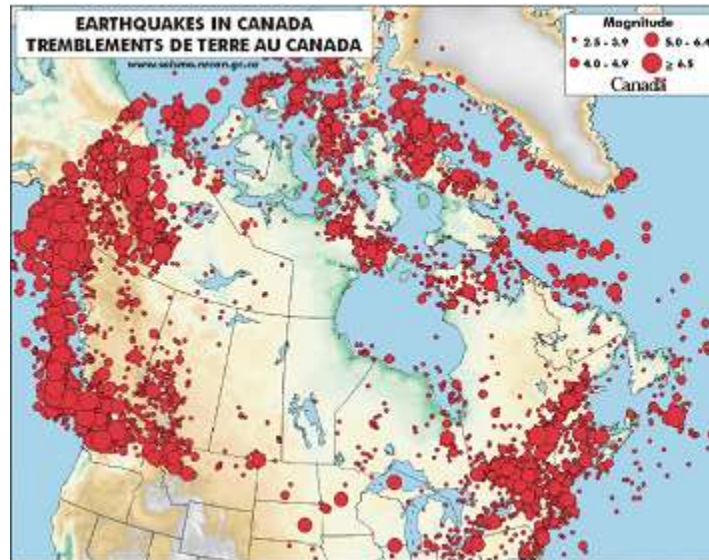
Under a moderate to low seismic activity, structures experience elastic deformations. Strong earthquakes may generate post-elastic response with significant inelastic deformations. Because the prevention of collapse is one of the fundamental design criteria, it is necessary to ensure that post-elastic deformations in all parts of the structure can be sustained without a significant decay in strength. This can be ensured by ductile behaviour. Ductility is defined as “inelastic deformability.” Structural elements constructed with ductile materials often possess sufficient inelastic deformability to absorb seismic induced energy. Therefore, structures constructed of ductile elements, such as structural steel and properly designed and detailed reinforced concrete tend to exhibit superior performance during earthquakes.

Ductile behavior can be achieved by promoting flexural response with properly designed and detailed plastic hinge regions. It is therefore accepted that during a low-probability strong earthquake, the inertia forces exceed the elastic load capacity of structural elements, creating plastic deformations in critical regions. Inelastic deformability in these regions can be achieved by appropriate confinement of the compression concrete through the proper arrangement of transverse and longitudinal reinforcement of sufficient quantity. Ductile inelastic response allows the structure to sustain large deformations while dissipating seismic-induced energy.

## **1.2 SEISMOLOGICAL BACKGROUND**

Earthquakes occur frequently in the world. Thousands of small (too small to be felt) earthquakes take place every year, while major earthquakes occur, on average, about once a year. Canada is a hotbed of seismic activity with an average of four to five sizeable earthquakes a day. In fact, seismologists estimate that more than 2,500 earthquakes are recorded each year across the country. While the majority can only be detected by seismographs, only about 100 of them with a magnitude of 3 or higher on the Richter scale are felt by humans. In the past 100 years, at least nine earthquakes have registered between 7 and 8 on the Richter scale in or near Canada. By looking at the Canadian Seismic Map shown in Fig 1.1, it can be seen that earthquakes can occur virtually in any region of Canada, though the most active parts of the country are the

western and southwestern regions of British Columbia, including Vancouver Island and the Queen Charlotte Islands, which are struck more than 200 times a year.

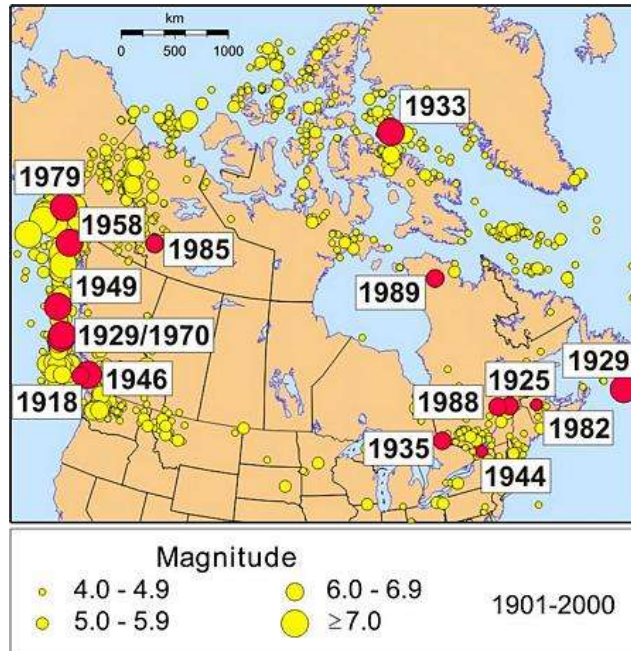


**Fig 1.1** Canadian Seismicity Map (Geological Survey of Canada, GSC)

Seismologists expect a major destructive earthquake along the Cascadian subduction zone, off the coast of Southern British Columbia. Exact timing of earthquakes cannot be predicted. But there is high probability that a moderate-to-strong earthquake will shake the current inhabitants of the Canadian West Coast during their lifetime.

Moderate earthquakes in the range of 6 on the Richter scale can also occur in Eastern and Northern Canada. In these regions, the Canadian Shield could carry the shock waves from an earthquake over a wide area, potentially causing significant damage. Evidenced by the 1988 Saguenay earthquake (Quebec), experts cannot rule out a major earthquake hitting Ottawa or Montreal.

The map in Fig. 1.2 displays the year, magnitude and location of major earthquakes in Canada during the 20th century. The majority of earthquakes occur along the west coast, with lesser number of events taking place in the east and the Maritimes.



**Fig 1.2** Major earthquakes in Canada in the 20<sup>th</sup> century  
(National Resources Canada)

### 1.3 SEISMIC ADEQUACY OF CONCRETE STRUCTURES IN CANADA

A large proportion of current infrastructure in Canada was built prior to 1970's, using seismically deficient building codes. These structures pose significant seismic risk and need to be upgraded

#### 1.3.1 PRE-1971 DESIGN PRACTICE

Since the 1971 San Fernando earthquake in the U.S., major changes in seismic design practices have been implemented in Canada and the United States. Similarly, the Japanese Building Code was revised in 1981 to set more stringent seismic design requirements, after the 1968 Tokachi-Oki and the 1978 Miyagi-ken Oki earthquakes. Of special interest for the current investigation is the column damage observed in bridges during past earthquakes. Fig 1.3 illustrates examples of bridge columns damage sustained during previous earthquakes.



1971 San Fernando Earthquake in California  
(Photo by U.S. Geological Survey)



1999 Chi-Chi Earthquake in Taiwan  
(Photo by Dr. David Lau)



1995 Kobe Earthquake in Japan  
(Photo by Dr. Michel Bruneau)



1994 Northridge Earthquake in California  
(Photo by Dr. Murat Saatcioglu)

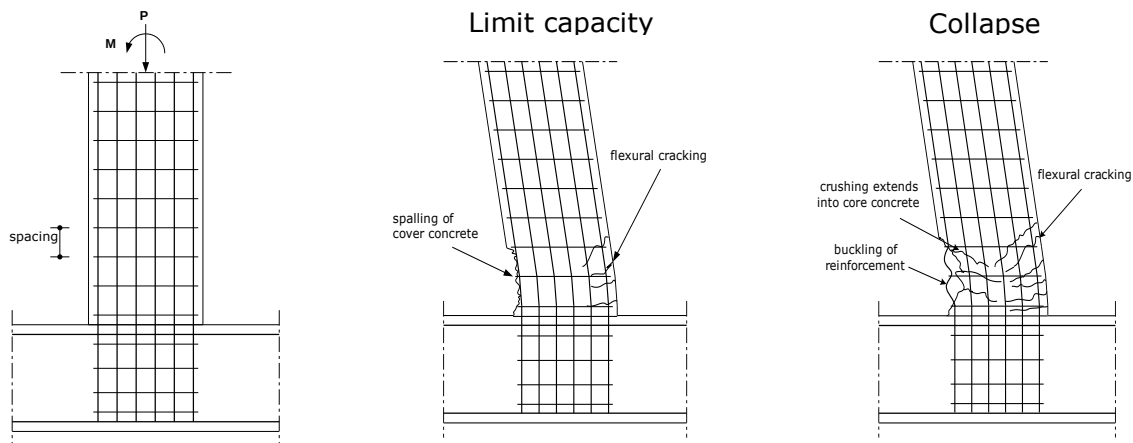
**Fig. 1.3** Damage sustained by bridge columns during previous earthquakes

Reinforced concrete columns constitute critical elements during strong earthquakes, providing strength and stability to the entire structure. The first storey building columns and bridge columns are blamed for many structural damage and failures in the past. A study on pre-1970's concrete bridge columns (Chai, Priestley and Seible, 1991) indicates the following structural inadequacies:



- Flexural strength is inadequate. A lateral force coefficient (ratio of seismic force to structural weight) less than 10% was used resulting in high ductility demands during sever earthquakes.
- Transverse reinforcement is typically 12.7 mm (# 4 U.S. bar size) ties spaced at 305 mm (12 in), resulting in lack of concrete confinement and diagonal tension capacity, producing brittle elements.
- Lap splice length of 20 times the bar diameter in potential plastic hinge regions, which is insufficient to develop the yield strength of longitudinal reinforcement.
- Footings and pile caps in some bridges consist of only bottom mesh reinforcement. Top steel and shear reinforcement are not provided.
- Joint regions are under-designed for shear.

Fig. 1.4 describes the lateral collapse mechanism of a column with insufficient transverse reinforcement. At displacement ductility ratios of 1 to 2, the spalling of cover concrete occurs in plastic hinge regions, where inelastic deformations concentrate. Unless the core concrete is well confined by closely-spaced transverse hoops or spirals, the crushing of cover concrete extends into the core, the longitudinal reinforcement buckles, and rapid strength degradation follows. This behavior is accelerated if the transverse reinforcement is lapped in the cover concrete, as is often the case in old construction. The hoops then loose their effectiveness at lap locations upon the spalling of concrete cover



**Fig. 1.4** Lateral collapse mechanism for an under-designed column in flexure

In Canada, structures built prior to 1971 suffer from the same deficiencies, as indicated above, posing a significant risk to public safety. A recent study by the Department of National Defense (Canada) indicates that approximately 7,200 out of 11,500 buildings owned and operated by the department are located in seismically active regions. Some of these buildings don't have the necessary reinforcing detailing required for seismic resistance, making them vulnerable to dynamic loads that deliver immense energy beyond their design limits. Another study by Public Works and Government Services Canada (PWGSC) reported that about 500 of its buildings had an average age of over 40 years, suffering from seismic deficiencies, with a replacement value of about \$4 billion. This excludes thousands of privately owned buildings and facilities with a similar situation. In order to mitigate seismic risk, the options are either to replace these structures or repair and retrofit them to resist seismic excitations. While the replacement of a significant portion of existing infrastructure is not feasible, retrofitting becomes the only economically viable approach to reduce such risk.

### **1.3.2 COLUMN RETROFITTING**

Concrete bridge piers built before the 1971 San Fernando earthquake, under the old design and construction practices, experienced severe seismic damage in California. While major changes in seismic design philosophies were implemented after this earthquake, seismic retrofitting of older structures, particularly reinforced concrete columns has been an important issue in many countries including Canada, USA and Japan. Retrofitting typically aims at increasing the available ductility of RC columns by enhancing concrete confinement and shear strength in potential plastic hinge regions. Improved confinement increases the ability of a column to withstand repeated cycles of loading beyond its elastic limit and prevents column failure due to the degradation of flexural capacity. De-bonding of longitudinal reinforcement lap-splices and formation of plastic hinges at regions of termination of longitudinal reinforcement can also be prevented by adequate confinement.

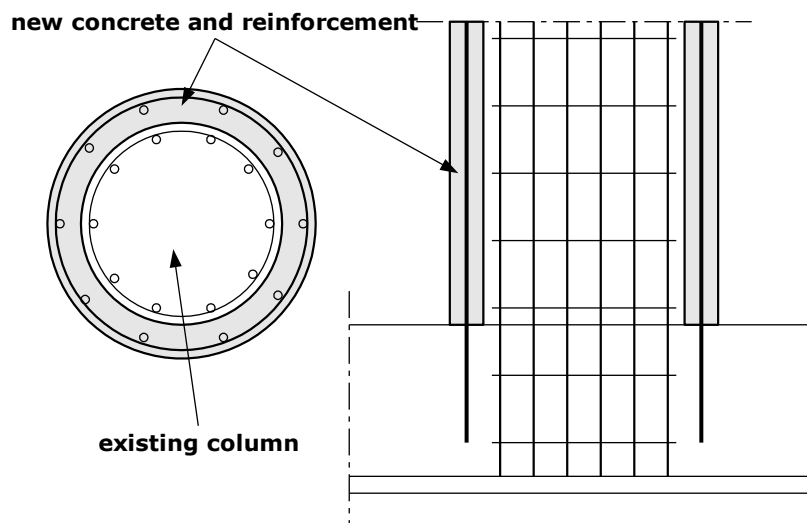
### 1.3.3 RETROFITTING METHODS

Reinforced concrete bridge columns can be retrofitted with one of the following four techniques:

1. Concrete Jacketing
2. Steel jacketing
3. Transverse prestressing (RetroBelt)
4. Fiber Reinforced Polymer (FRP) Jacketing

#### 1.3.3.1 CONCRETE JACKETING

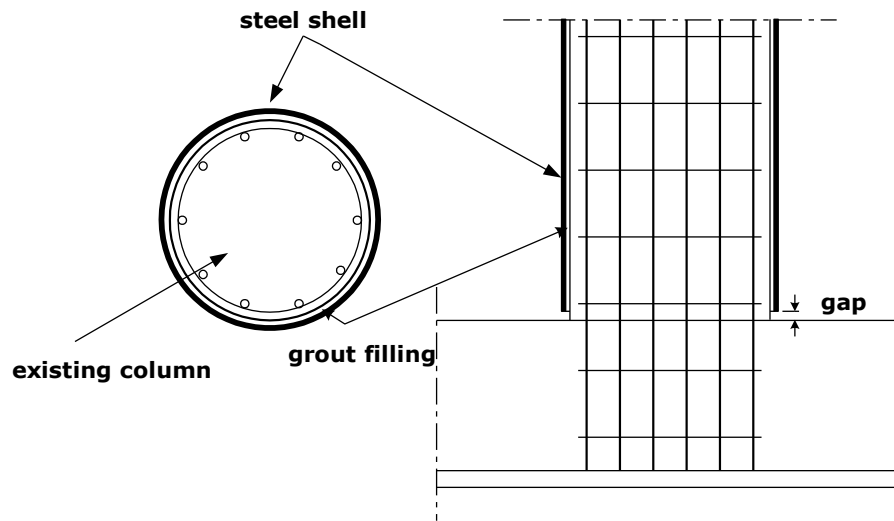
A concrete jacket consists of a thick layer of reinforced concrete cast around a column (Fig. 1.5). Closely spaced longitudinal and transverse reinforcement is added in the new layer of concrete, improving strength and ductility. The existing cover concrete is removed first to expose the main reinforcing bars. Chipping away the concrete cover roughens the surface which improves bond between the old and the new concretes. U shaped steel links are then welded to the exposed bars. Additional bars are then welded to the U shaped links to form the longitudinal reinforcement. Stirrups are added as required and concrete is poured after the erection of timber formwork. This is one of the most commonly applied methods of repair and strengthening of concrete columns. Apart from welding, it does not require skilled labour. Concrete jacketing has been commonly used in Japan, mostly for enhancing flexural strength.



**Fig. 1.5** Concrete Jacketing of a circular section column

### 1.3.3.2 STEEL JACKETING

A steel jacket usually consists of two half shells of steel placed around a column, with a clearance of about 10mm to 25 mm, and welded together, as illustrated in Fig. 1.6. The gap between the jacket and the column is filled with pure cement grout. A space of about 50 mm is left between the jacket and the joining member, or the footing, to avoid the possibility of loads being carried by the jacket at large drift angles, which would cause local buckling of the jacket. The jacket provides a passive confinement action. Lateral confining pressure is induced in the concrete as it dilates laterally under high axial strains in the compression zone, due to the hoop strength and stiffness of the jacket. The jacket can be considered as equivalent to continuous hoop-reinforcement.



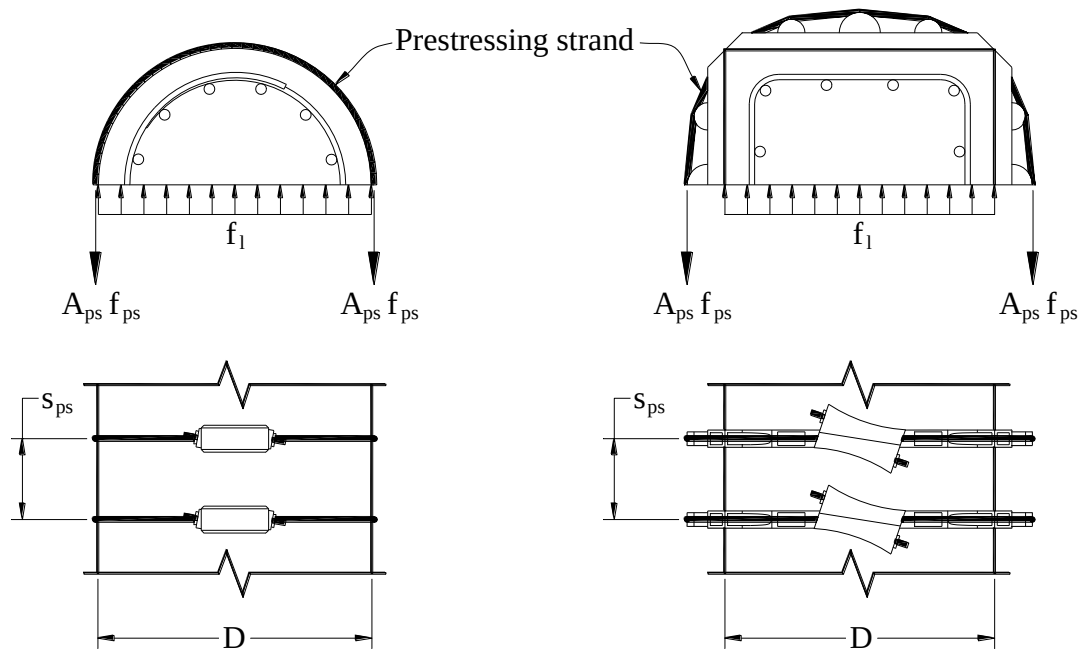
**Fig. 1.6** Steel jacketing of a circular section column

### 1.3.3.3 EXTERNAL PRESTRESSING STRANDS (RETROBELT)

This technique was developed at the University of Ottawa through experimental research that involved tests of full size concrete columns with different cross-sections, aspect ratios, and reinforcement details (Saatcioglu et al 2000, Mes and Saatcioglu 2006, Saatcioglu et al. 2002, Saatcioglu and Yalcin 2003). The basis for the method is to apply active confinement pressure using external prestressing strands (in the form of external hoops). The hoop tension is transferred to the column directly by the strands, in the case

of circular columns, and through spreader frames in the case of square and rectangular columns. The technique is illustrated in Fig. 1.7. It was experimentally shown to provide significant improvements in column deformability in flexure, shear and splice-deficient bridge columns (Yalcin 1998, Mes 1999, Beausejour, 2000).

Although steel and concrete jackets were widely used since the 1971 San Fernando Earthquake, external prestressing is less expensive and less labour intensive. It does require surface treatment, in the form of shotcreting to protect the strands against external effects such as corrosion, fire and vandalism.



**Fig. 1.7** RetroBelt® Retrofitting System

#### 1.3.3.4 FIBER REINFORCED POLYMER COMPOSITES

Conventional steel and concrete jacketing procedures, as well as the recently developed external post-tensioning technique, have some drawbacks. These include;

- Heavy weight, resulting in extra mass for seismic forces, though relatively small in comparison with total structural mass.
- Increased stiffness of structural member, leading to a reduced period and increased seismic forces.

- Labor intensive.
- Corrosion of steel and associated durability problems.
- May involve intrusive construction techniques resulting in the interruption of traffic.
- Difficulty in installation in narrow and hard-to-reach places.
- Reduced aesthetics.

An alternative to conventional construction techniques and construction materials is the use of advanced composites, which may eliminate the majority of the above disadvantages. Composite materials made of carbon and glass fibers, impregnated in polymeric resins, also known as fiber-reinforced polymers (FRP), have emerged as an alternative to these traditional materials and techniques. FRPs can be up to six times stronger than steel, has only 20% of the weight, and is non-corrosive and electro-magnetically neutral. They are made of small-diameter glass, carbon, or aramid, fibers, saturated in a resin matrix. The primary load-carrying elements within the composite are the fibers. The fiber has strong influence on the mechanical properties of the composite such as strength and elastic modulus. The resins provide a mechanism for the transfer of load among the fibers. It also protects the fiber from abrasion and other environmental and chemical attacks. The fiber and the composite can be oriented in a single direction or several directions to optimize structural performance.

### **Advantages of FRP over traditional retrofitting techniques**

- Fibers are non-corrosive material, making them an ideal choice in a harsh weather environment.
- They possess high strength under short term, long term and repeated loading.
- Owing to low specific weight ( $17\text{-}18 \text{ kN/m}^3$  depending on the fiber to matrix ratio) fibers:
  - Have low transportation costs
  - Add less weight to the structure
  - Are easy to apply even in restricted spaces

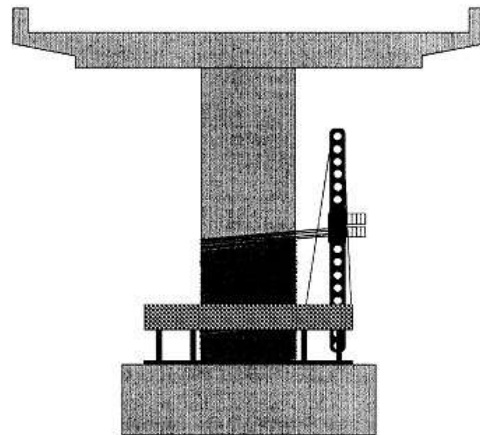
- Have no need for scaffolding to support the strengthening strips or fabrics during hardening of the adhesive
- Have no limitation in length
- Due to their fabric thickness (1 to 1.4 mm):
  - Do not result in structural height or usable space
  - Strips can be crossed
  - Applications are easy to cover, thus fulfilling aesthetic requirements
- Both active and passive confinement are possible with composites
- Even though carbon fibers are relatively expensive as a material, this technique is considered an economical solution considering the entire cost of strengthening and future maintenance.

The use of externally bonded FRP composite strips and sheets provides an efficient method for repair and strengthening of structural element, including columns, walls, beams, girders, slabs and decks. In contrast to externally bonded steel plates, the application of FRP composites is extremely easy (even in confined spaces), does not add weight to the structural element being repaired, and can be completed with minimum disruption to traffic or occupancy. FRP's can be applied on columns by following one of the three procedures; i) wrapping, ii) filament winding, and iii) using prefabricated shells.

**Wrapping:** In situ wrapping has been the most common technique for column retrofitting using FRP composites. Experimental studies have demonstrated that FRP wrapping of columns with one or more layers provides a confining action on concrete which enhances both the strength and ductility of columns. In this method, unidirectional fiber sheets or woven fabric sheets are impregnated in polymer resins and wrapped around columns in a wet-lay-up process, with main fibers oriented in the hoop direction. A column can be fully wrapped with FRP sheets in single or multiple layers, but it can also be partially wrapped using FRP straps in continuous spirals or discrete rings. The compressive strength enhancement of concrete due to external wrapping of FRP was first demonstrated by Fardis and Khalili (1981, 1982). This concept was first applied to the strengthening of actual reinforced concrete columns in Japan in the mid-1980s (American

Concrete Institute 1996). There have been many reports of the application of this technique. Fabrics can be unidirectional or multidirectional. Resin plays a dual role of impregnating the fibers and ensuring their bond to concrete surface. There have been many reports of field applications for retrofitting bridge and building columns (e.g., American Concrete Institute 1996; Neale and Labossiere 1997; Tan 1997).

**Filament Winding:** The principle of filament winding is similar to wrapping, except that the filament winding technique uses continuous fiber strands instead of sheets/straps so that the winding can be processed automatically by means of a computer-controlled winding machine. This process can produce an FRP jacket with controlled thickness, fiber orientation, and volume fraction. The idea of confining concrete by winding continuous resin-impregnated fiber strands was first mentioned by Fardis and Khalili (1981). The first winding machine for column retrofitting was developed in Japan in mid 1980s (ACI 1996). The wrapping is performed by a patented machine known as Robo-Wrapper (Fig. 1.8). Currently there are three versions of the machine available for bridge retrofit work depending on the size of the columns being wrapped. The material used is manufactured by Thiokol Corporation and is designated "Akso Fortifil 50K/UF3325-90". It is made of carbon fibers pre-impregnated with epoxy resin under factory-controlled conditions. Both the amount of fibers and their pitch is automatically controlled according to the requirements.

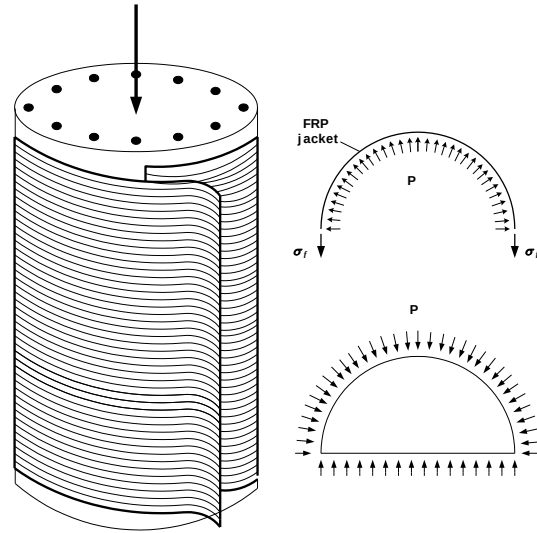


**Fig. 1.8** Automatic wrapping (Fib 2000)



**Prefabricated Shell Jacketing:** Existing reinforced concrete columns can also be retrofitted using prefabricated FRP shells. The shells are fabricated under controlled conditions using fiber sheets or strands, with impregnation by resins before field installation. They can be fabricated in half-circles or half-rectangles (Nanni and Norris 1995), in circles with a slit, or in continuous rolls (Xiao and Ma 1997), so that they can be opened and placed around the columns. To achieve effective FRP confinement, full contact between the column and the FRP shell is essential. This contact can be ensured by either bonding the shell to the column using adhesives (Xiao and Ma 1997), or injecting shrinkage-compensated cement grout or mortar into the space between the shell and the column (Nanni and Norris 1995). An interesting application of prefabricated shells is for formwork in modifying column shapes as part of a strengthening measure. This has recently been discussed by Teng et al (2002), who suggested that square or rectangular columns can be strengthened by reshaping them into circular or elliptical columns in which a prefabricated FRP shell is used to provide both the permanent formwork and the required confinement after the curing of concrete.

Before FRP is applied using any of the above techniques, the column should be properly prepared to provide a hard, dry, and clean surface. Any damaged or deteriorated parts should be removed and the surface should be patched with epoxy putty as needed. Wrapping requires that square column corners be rounded. This is necessary to reduce any detrimental effect of sharp corners on the tensile strength of FRP (stress concentration effect) and to enhance the effectiveness of confinement. Whether wrapping or prefabricated shell jacketing is used, a sound confinement can be achieved by extending the end of the final layer of the FRP to form a sufficient overlap, as indicated in Fig. 1.9. The confinement obtained through these methods generates “passive confinement pressure,” activated only by transverse dilation of compressed concrete (Fig. 1.9).



**Fig. 1.9** Confining columns with FRP, taking advantage of the transverse dilation of the compressed concrete core under axial load

#### 1.3.4 FRP STAY-IN-PLACE FORMWORK

The application of FRP to reinforced concrete columns is not limited to retrofitting existing structures and may be used for new construction. Corrosion of steel reinforcement in concrete is the most prevalent form of deterioration in reinforced concrete structures in Canada. The severe environmental conditions and the use of deicing salts lead to serious concerns regarding structural integrity of bridge and building infrastructure. Cracking and spalling of concrete in bridges, parking garages and other structures, resulting from the expansion of corroding reinforcement in concrete is a widespread problem. Because of its superior weather resistant characteristics, FRP's provide ideal protection against steel corrosion (Lee et al. 2000; Pantazopoulou et al. 2001; Masoud and Soudki 2002), while also providing concrete confinement for improved deformability. Fig. 1.10 shows a concrete bridge column designed according to seismic provisions of new codes with continuous spiral reinforcement. However, the column is badly deteriorated due to steel corrosion, exposing steel and spalled cover concrete. Prefabricated FRP casings can be used as “stay-in-place formwork” with multiple advantages of corrosion protection, improved seismic performance, and savings in formwork.



**Fig. 1.10** Steel corrosion and spalling of cover concrete on a concrete bridge column

## 1.4 RESEARCH NEEDS

Extensive experimental and analytical research has been conducted on seismic retrofitting of reinforced concrete columns by means of conventional materials and techniques. The application of FRP jacketing, as a new material, is currently being investigated. While certain aspects of FRP jacketing have been established, including its effectiveness as confinement and shear reinforcement in circular columns, many aspects remain to be established. There is need for further research to i) establish the level of confinement attained in square and rectangular columns, ii) establish the effectiveness of FRP jacketing in providing lateral clamping force in reinforcement splice regions, iii) establish the characteristics of FRP in shear, iv) develop design methodologies that can be codified, and v) develop design and construction techniques for FRP stay-in-place formwork for new construction.

## 1.5 OBJECTIVES

The primary objective of the proposed research project is to develop effective and reliable techniques for earthquake resistant reinforced concrete columns wrapped with carbon FRP (CFRP) sheets for use in existing structures and CFRP stay-in-place formwork for new construction. The objective includes the generation of experimental

data to demonstrate the performance of reinforced concrete bridge columns encased in CFRP jackets when subjected to simulated seismic loading. It also includes the development of design techniques suitable for codification and use in practice.

## **1.6 SCOPE**

The scope of proposed research includes the following:

- Design of seven full-scale reinforced concrete bridge columns for construction and testing under simulated seismic loading.
- Construction and retrofitting of four seismically deficient bridge columns using externally bonded CFRP sheets and the wet lay-up method.
- Preparation of three CFRP tubes with circular and square cross-sections in the form of CFRP stay-in-place formwork, with and without internal crossties.
- Construction of three bridge columns using the above CFRP stay-in-place formwork.
- Instrumentation of all column specimens for strain, displacement and rotation measurements.
- Preparation of the required test set-up.
- Column tests under constant axial compression and incrementally increasing lateral deformation reversals.
- Evaluation of test data.
- Analyses of columns for flexure and shear, including the construction of analytical force-deformation relationships for verification.
- Development of a displacement based design procedure for seismic retrofitting reinforced concrete columns with CFRP sheets and use of CFRP stay-in-place formwork for new bridge columns.

## **1.7 EARLIER RESEARCH AT THE UNIVERSITY OF OTTAWA ON USE OF CFRP SHEETS FOR COLUMN JACKETING**

The proposed research forms the second phase (Phase II) of an ongoing research project at the University of Ottawa, aimed at developing design and construction techniques for concrete columns externally reinforced with FRP composites. Phase I was conducted by

Elnabelsy (2000) and included three full-size concrete bridge columns, retrofitted with CFRP wrapping. The columns consisted of circular and square columns with and without spliced reinforcement, tested under simulated seismic loading. Phase II of the project form the scope of current research and includes tests of additional seven columns, as well as analytical research. Because the proposed research is the continuation of the earlier phase, previous results are briefly summarized below.

### 1.7.1 PHASE I - SPECIMEN DETAILS

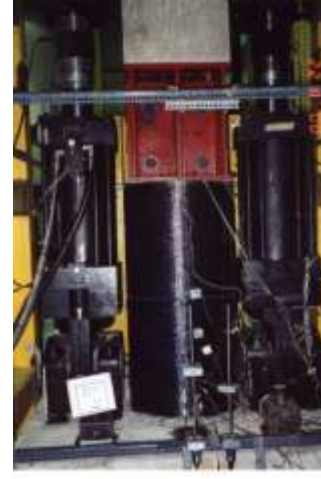
Phase I consisted of a flexure dominant circular column (**BR-C8-1**), a flexure dominant square column (**BR-S5-1**) and a shear dominant circular column (**BR-C10-1**). The specimens are illustrated in Fig. 1.11 during testing.



BR-C8-1 (4 plies)



BR-S5-1 (5 plies)



BR-C10-1 (4 plies)

**Fig. 1.11** Retrofitted Columns in Phase I

The specimens represented either the full length of bridge columns between the fully fixed foundation and a hinge support at the top or the portion of a fixed-fixed column between the foundation and the point of inflection. Each column had a cross-sectional dimension that varied between 500 mm and 610 mm, as shown in Table 1.3. Twelve No. 20 bars were used as longitudinal reinforcement. No. 10 hoops were used at 300 mm spacing, with the first hoop placed at 75 mm from the column-footing interface. The circular columns had circular hoops with overlapping ends. The square hoops had 135°

hooks at the ends. All reinforcing steel was of grade 400MPa. The longitudinal reinforcement was spliced near the base with a lap length of 400 mm, corresponding to 20 times the bar diameter. CFRP sheets were used to wrap columns with fibres in circumferential direction. The specimen details and number of CFRP layers used are summarized in Table 1.1.

**Table 1.1** Properties of Phase I Specimens

| <b>Column</b> | <b>Cross-Section</b> | <b>Sectional Dimension (mm)</b> | <b>Shear Span (m)</b> | <b>No. of CFRP Plies</b> |
|---------------|----------------------|---------------------------------|-----------------------|--------------------------|
| BR-C8-1       | Circular             | 508                             | 2.0                   | 4                        |
| BR-S5-1       | Square               | 500 × 500                       | 2.0                   | 5                        |
| BR-C10- 1     | Circular             | 610                             | 1.5                   | 4                        |

### 1.7.2 PERFORMANCE OF PHASE I SPECIMENS

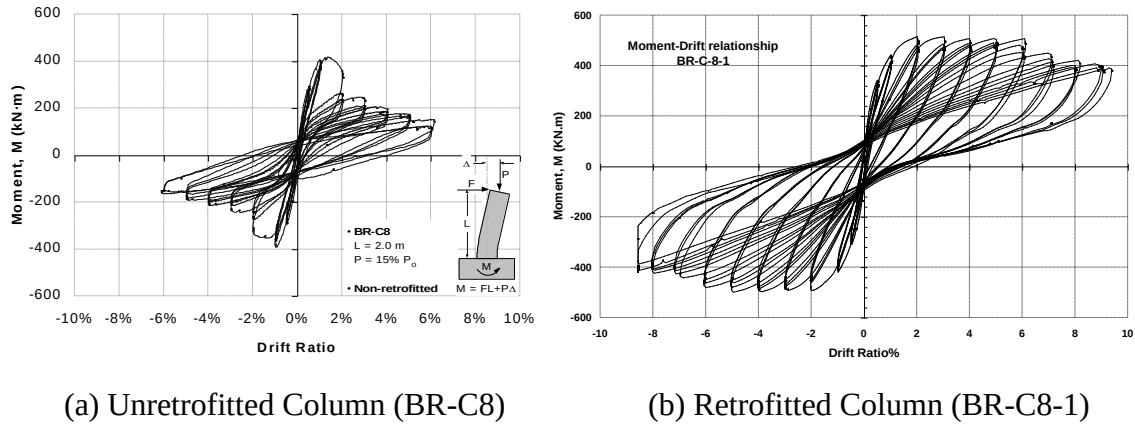
#### **Long Circular Column (BR-C8-1)**

The column was 1.7 m high, with an effective height (shear span) of 2.0 m because of a 275 mm loading beam placed on the column. It had a 508 mm circular section and 12-No.20 bars placed with a 50mm clear concrete cover. Retrofitting was carried out using four plies of CFRP sheets, covering the critical bottom portion of the column. The column was able to withstand lateral force capacity up to 8% drift without a significant strength decay, compared with 1% lateral drift observed in the control specimen BR-C8, tested earlier by Beauséjour (2000) as part of another bridge retrofit research program at the University of Ottawa. Fig. 1.12 shows moment-drift hysteretic relationships for the two companion columns.

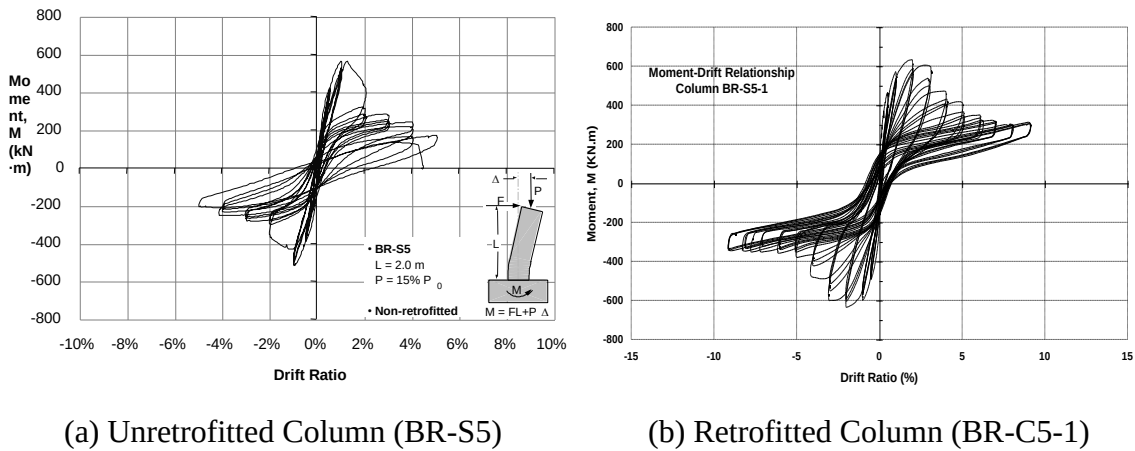
#### **Long Square Column (BR-S5-1)**

The specimen BR-S5-1 was companion to the long circular column discussed in the earlier section, except for the square cross-sectional shape. The column height was also equal to 1.7 m high, with a shear span of 2.0 m. The cross-sectional dimension was 500 mm with 12-No. 20 longitudinal bars placed with a clear cover of 45 mm. Five plies were used to wrap the column. The drift capacity of the retrofitted column was 3%

compared to 1% drift capacity for the control column BR-S5, tested by Beauséjour (2000). However, the improvement in column deformability was much less than that observed in companion circular column. Fig. 1.13 shows moment–drift hysteretic relationships for both the control (BR-S5) and retrofitted (BR-S5-1) specimens. As it can be seen, the hysteretic relationship for the retrofitted column shows a pinched behaviour, signifying the slippage of lapped bars.



**Fig. 1.12** Hysteretic Relationship for Columns BR-C8 and BR-C8-1

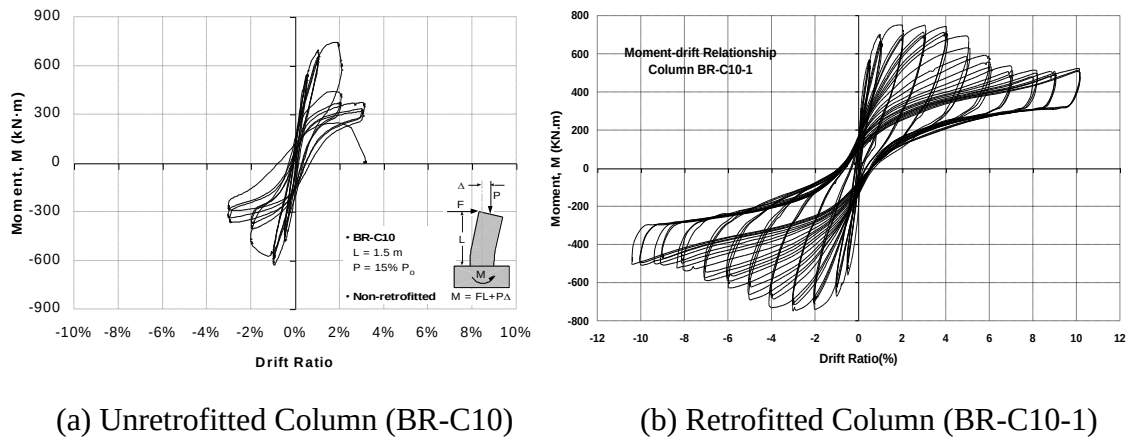


**Fig. 1.13** Hysteretic Relationship for Columns BR-S5 and BR-S5-1

### Short circular column (BR-C10-1)

The specimen BR-C10-1 was companion to BR-C8-1, except for its reduced length of 1.2 m, with a shear span of 1.5 m, which promoted shear dominant behaviour. The column had a 610 mm circular cross-section with 12-No.20 longitudinal bars and a clear

concrete cover of 55 mm. Four plies of CFRP sheets were used to retrofit the column, covering the entire column height. The drift capacity of the column was 5% compared to 1% for the control column BR-C10 tested by Beauséjour (2000). Fig. 1.14 shows moment–drift hysteretic relationships for both unretrofitted BR-C10 and retrofitted BR-C10-1. The results indicate a somewhat reduced deformability of BR-C10-1 with a shorter shear span, as compared with BR-C8-1 with a longer shear span, because of the presence of higher shear stress reversals in the former column. Furthermore, the excessive pinching observed in the hysteretic relationship for BR-C10-1 is an indication of high shear force reversals in the column.



**Fig. 1.14** Hysteretic Relationship for Columns BR-C10 and BR-C10-1

## 1.8 LITERATURE REVIEW

Review of previous research involving FRP wrapping of columns and FRP stay-in-place formwork for improved seismic performance of columns are presented below in two separate sections.

### 1.8.1 PREVIOUS RESEARCH ON FRP WRAPPING OF CONCRETE COLUMNS AS A SEISMIC RETROFIT STRATEGY

There has been significant research conducted on seismic retrofitting of reinforced concrete columns. Increasingly more research is being carried out on the effectiveness of FRP as a seismic retrofit material. Katsumata et al. (1987, 1988) were among the first who proposed the use of FRP composites for seismic retrofitting. They tested five



circular and 10 rectangular quarter-scale columns with and without laterally wound carbon fiber strands to study their performance under combined axial and cyclic lateral loads. Later, Matsuda et al. (1990) identified two retrofitting methods for RC bridge columns using FRP composites: (a) strength-oriented and (b) ductility-oriented retrofitting. In the first method, FRP plates were longitudinally bonded to increase the flexural strength of columns (i.e., surface bonded longitudinal FRP); in the second method, FRPs were wrapped around columns with fiber orientation (or main fiber orientation) in the hoop direction to enhance its ductility as proposed by Katsumata et al. (1987, 1988). Both methods were found to improve the energy absorption capacity of columns. In recent years, the technique of strengthening RC columns using FRP composites has been used increasingly to replace the steel jacketing (Ballinger et al. 1993; Saadatmanesh et al. 1997; Seible et al. 1997). The most common form of FRP column strengthening involves external wrapping of FRP sheets/straps. The strengthening is based on well-established concepts of lateral confinement (Richart et al. 1928, 1929; Ahmad and Shah 1982; Mander et al. 1998). Many studies were conducted on compressive strength and stress-strain behavior of FRP-confined concrete (e.g., Fardis and Khalili 1981; Ahmad et al. 1991; Harmon and Slattery 1992; Demers and Neale 1994; Howie and Karbhari 1994; Nanni and Bradford 1995; Karbhari and Gao 1997; Mirmiran et al. 1998a; Demers and Neale 1999; Miyauchi et al. 1999; Purba and Mufti 1999; Saafi et al. 1999; Toutanji 1999; Rochette and Labossiere 2000; Xiao and Wu 2000; Zhang et al. 2000). These studies have shown that FRP-confined concrete behaves differently than steel-confined concrete (Mirmiran et al. 1997; Samaan et al. 1998; Saafi et al. 1999; Spoelstra and Monti 1999). Therefore, the design recommendations for steel-confined concrete columns cannot be applied to columns confined with FRP, despite the apparent similarity between these types of retrofit methodologies.

In California, more than 500 bridge piers were wrapped with advanced composite materials, immediately after the 1994 Northridge earthquake (Seible et al. 1995, Xiao et al. 1995). Similar programs were applied in Japan (Hoshikuma and Unjoh 1997, JSCE 1995). In Europe, where notable interest exists (FIB 2001), the subject is still in an

interlocutory phase, mainly because of the lack of established and accepted design rules which slow down the process of promoting FRP as a ‘recognized’ construction material.

To give an overview of the state-of-the art and for better understanding of the behavior of FRP retrofitted seismically deficient columns, selected papers are summarized hereafter.

***Saadatmanesh, Ehsani and Jin (1996)***

The objective was to retrofit old columns with FRP composite straps to increase column ductility. A total of five  $\frac{1}{5}$ -scale models were built based on highway bridge design practices before 1971. All the specimens were circular with a 12" (305 mm) diameter. The longitudinal reinforcement was continuous in two columns and was spliced at the bottom of the columns in the remaining three columns over a length of 20 times the bar diameter. The strengthened columns were wrapped with six layers of FRP containing unidirectional glass fibres for a total thickness of 5 mm (0.2 in). The fibres were aligned in the circumferential direction for confinement. The gap between the column and the straps was filled with pressurized epoxy resin to ensure the development of lateral pressure. The columns were subjected to an axial load of 445 kN (100 kips) corresponding to approximately 16%  $A_g f'_c$ . The lateral displacement history applied consisted of two complete cycles for each ductility ratio of  $\mu = 1, 1\frac{1}{2}, 2, 3, 4$ , etc. The non-retrofitted columns with spliced reinforcement sustained lateral loads up to a low ductility level of  $\mu = 1\frac{1}{2}$  (about 2.2% drift). Companion columns with continuous reinforcement sustained  $\mu = 3$  (approximately 3.7% drift). The retrofitted columns developed stable hysteresis loops up to  $\mu \approx 6$  (roughly 6½% drift). The researchers concluded that FRP wrapping in potential plastic hinge regions increased strength and ductility.

***Seible, Priestley, Hegemier and Innamorato (1997)***

The authors Introduced general guidelines to retrofit reinforced concrete columns with carbon fiber jackets. Seven bridge columns were tested. Each column was subjected to a constant axial load and incrementally increasing lateral deformations. The lateral loading scheme consisted of three cycles at each ductility ratio,  $\mu = 1, 1\frac{1}{2}, 2, 3, 4, 6, 8, 10, 12$

etc., until failure. Test results were compared with three previously tested columns that were retrofitted with steel jackets. Two of the columns tested had a 610 mm  $\times$  406 mm rectangular cross-section with 2.4 m column height. The shear span of these columns was 1.2 m because the end conditions were fixed-fixed. The column aspect ratio was chosen to obtain a shear dominant behaviour. The longitudinal reinforcement was continuous from the footings up to the top of the columns. An axial load of 507 kN was applied corresponding to 5.9%  $A_g f'_c$ . The non-retrofitted column was able to sustain lateral loads up to a displacement ductility ratio of  $\mu = 3$  (2.7% drift), compared with  $\mu = 10$  (7.4% drift) sustained by the column retrofitted with a carbon fibre jacket. The second pair of rectangular columns tested had a 730 mm  $\times$  489 mm cross-section and 3.7 m height. They were designed to have a flexure dominant behaviour. The columns had continuous longitudinal reinforcement without splices and were subjected to an axial load of 1780 kN (14.5%  $A_g f'_c$ ). The control column (non-retrofitted) failed after a ductility ratio of  $\mu = 3$  (2½% drift), while the column retrofitted with a carbon fibre jacket maintained its lateral load resistance up to  $\mu = 7$  (5.7% drift). Three other columns were built with a 610 mm diameter circular cross-section and 3.7 m height. The longitudinal reinforcement was spliced at the bottom of the column over a length equal to 20 times the bar diameter. Each column was tested under a constant axial load of 1780 kN (17.7%  $A_g f'_c$ ). One of the columns with carbon fibre jacket exhibited slippage (de-bonding) of lap splices after a displacement ductility ratio of  $\mu = 5$  (4½% drift). It was subsequently realized that the actual jacket thickness was 20% less than the design jacket thickness. Therefore, the researchers conducted another test with a slightly higher FRP thickness than that required in their design. The results of improved and the column was able to sustain a ductility ratio of  $\mu = 10$  (6.8% drift) without a significant strength decay. It was concluded that a composite jacket was just as effective as conventional steel jacket in improving the performance of reinforced concrete columns with poor details.

***Saadatmanesh, Ehsani and Jin (1997)***

In this study, a method, utilizing FRP straps to retrofit rectangular bridge columns was investigated. The aim was to evaluate the effectiveness of FRP straps on columns with inadequate splicing or insufficient transverse reinforcement. Five scaled models were

built, reflecting the pre-1971 design practice. The rectangular cross-section was 241 mm  $\times$  368 mm and the height was 1.8 m. An axial load of 15%  $A_g f'_c$  was applied to the columns. The lateral loading consisted of two reversed cycles for each ductility level of  $\mu = 1, 1\frac{1}{2}, 2, 3, 4, 5, 6$  etc., until failure. Two specimens had lap splices of 254 mm long at the bottom of the column, corresponding to a length of 16 times the bar diameter. The non-retrofitted column failed due to the slippage of splices once the ductility level reached  $\mu = 1\frac{1}{2}$  (2% drift). The other column was strengthened with eight plies of FRP in the potential plastic hinge region. The straps had a rectangular shape. A pressurized epoxy resin was introduced between the FRP straps and the concrete cover to ensure proper confinement. This column sustained lateral loads up to  $\mu = 5$  (6.4% drift). Three additional specimens were designed as shear dominant columns, without the reinforcement splices. The as-built specimen resisted loads up to a ductility ratio of  $\mu = 2$  (2.8% drift) and failed due to shear, as anticipated. The other two columns were strengthened either with rectangular shaped straps or with oval shaped straps providing passive pressure in both cases. These specimens were wrapped with eight plies in the hinging region and three plies everywhere else for increased shear capacity. Test results showed stable load-displacement hysteresis loops up to  $\mu \approx 6$  (7½% drift) without significant structural deterioration. The tests were stopped because the actuator reached its maximum stroke.

It was concluded that columns strengthened effectively with FRP straps increased strength, ductility and energy dissipation. No major difference was observed between the behaviours of rectangular and oval shaped columns. Additional tests were recommended to investigate the significance of modifying the cross-section for improved confinement.

### ***Xiao and Ma (1997)***

The objective was to evaluate the efficiency of prefabricated composite jackets in retrofitting reinforced concrete bridge columns. The advantages of using prefabricated jackets were stated as; better quality control and curing of the composite jacket in the plant, and quicker installation in the field. Three specimens with a 610 mm circular cross-section and 2.44 m height were constructed. The longitudinal bars were spliced over a length of 20 times the bar diameter near the bottom of the column. The first specimen

was tested as-built and then repaired with four layers of prefabricated composite jacket within the bottom half. Each layer was made of unidirectional glass fibre sheet, arranged in such a way that 90% of the fibres were in the circumferential direction and 10% in the longitudinal direction. The thickness of each layer was approximately 3.2 mm. The second specimen was retrofitted with three layers over a 1220 mm segment of column and the fourth layer over 610 mm length, whereas the last column had five layers on the first 1220 mm and three layers on the next 991 mm segment. The researchers indicated that the upper portion retrofit was mainly for construction demonstration, rather than required by design. To simulate the typical axial load of 5% of the capacity, used for multi-column bent bridges in California, the columns were subjected to  $5\% P_o$  ( $5\frac{1}{4}\% A_g f'_c$ ). The lateral loading consisted of three cycles at each displacement ductility ratio of  $\mu = 1, 1\frac{1}{2}, 2, 3, 4, 6, 8$ , etc., until failure. The tests showed enhancements in hysteretic response and ductility. The retrofitted columns maintained stable hysteresis loops up to the first cycle of a ductility ratio  $\mu = 6$  (near 4% drift), whereas the control column could barely withstand its capacity at  $\mu = 1$  (below 1% drift) with a lower flexural strength. The repaired column showed a slight increase in flexural capacity, and failed at  $\mu = 4$  (3% drift).

#### ***B.K. Purba and A.A. Mufti (1999)***

The purpose of this study was to investigate the behaviour of concrete columns confined with FRP jackets. The researchers used a stress-strain model for confined concrete developed by Saadatmanesh et al. (1994) based on the model proposed by Mander et al. (1988). Although this model was originally developed for conventionally reinforced concrete columns it was applied to steel as well as FRP jacketed columns. Three circular columns with 190 mm cross-sectional dimension and 788 mm height were tested under monotonically increasing concentric loading. The specimens were designated as; C-1, H2-1, and H2-2. Specimen C-1 was the control specimen and the remaining two were wrapped with two layers of transverse carbon fiber sheets. Specimen C-1 and H2-1 were each instrumented with axial and circumferential gauges at mid height. Column H2-2 was instrumented with two additional axial gauges to verify the absence of bending and to assess the uniformity of confining stresses, for which the three gauges were placed at

120° from each other. The results obtained are summarized in Table 1.2. It is obvious that the increase in ultimate load and axial strain in retrofitted specimens were highly improved compared to the control specimen. The study also showed that the theoretical results for specimen H2-2 agree with experimental ones, whereas the results for specimen H2-1 vary somewhat. The authors justified such variation by the fact that axial strains for specimen H2-2 are more representative of the actual axial strain since it is the average of 3 gauges. Another justification was attributed to the degree of tightness of the CFRP. As externally applied CFRP sheets are affected by both the amount of resin and manual pressure applied during the lay-up process, it was possible to have a loose CFRP jacket for this column. A loose jacket would require greater outward expansion of the core concrete than a tight jacket before confinement stresses could be developed.

**Table 1.2** Specimens Tested by Purba and Mufti (1999)

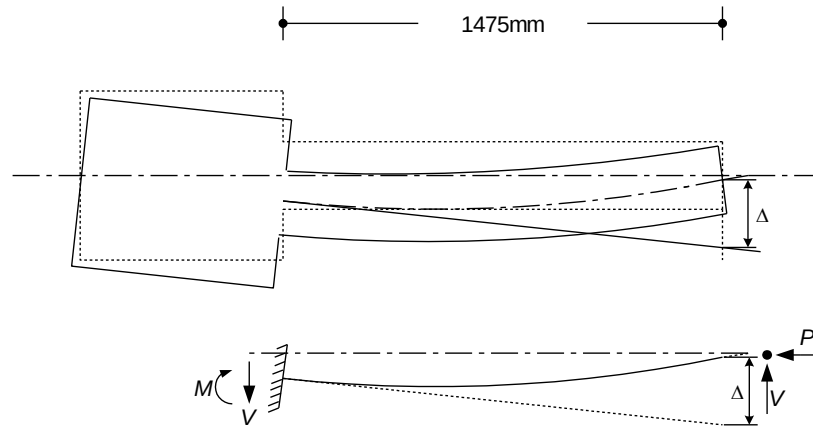
| <b>Specimen</b>                                | <b>C-1</b> | <b>H2-1</b> | <b>H2-2</b> |
|------------------------------------------------|------------|-------------|-------------|
| Ultimate load (kN)                             | 772        | 1537        | 1416        |
| Increase in ultimate load (%)                  |            | 99          | 83          |
| Ultimate axial strain (micro strain)           | 1987       | 5757        | 4994 (avg.) |
| Increase in axial strain (%)                   |            | 190         | 151         |
| Ultimate Circumferential strain (micro strain) | 394        | 6648        | 4451        |

The authors reported the following conclusions:

- FRP jacketing has positive effects for enhanced behavior, durability and in the case of prefabricated jackets, ease of construction.
- The theoretical and experimental results indicate a remarkable confining effect of FRP jackets as evidenced by the increase in ultimate strength and ductility.
- Extensive research and experimentation is needed in the area of FRP jacketing before the behavior of such columns can be accurately predicted and field application can be made.

**R.D. Lacobucci, S.A. Sheikh, and O. Bayark (2002)**

This research was conducted to investigate the prospect of strengthening deficient columns, as well as repairing damaged square columns with CFRP Jackets. Eight specimens (305x305x1473 mm) with non-seismic transverse steel detailing (representing those built before 1971) were tested under constant axial load and lateral displacement cycles, simulating earthquake forces.



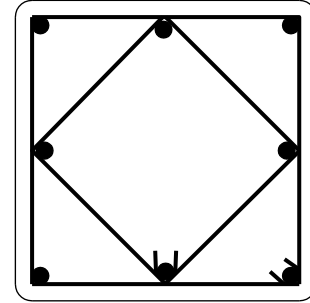
**Fig. 1.15** Idealization of Specimens by Lacobucci et al. (2002)

Three variables were analyzed: i) number of CFRP layers, ii) presence of column damage and iii) the level of applied axial load. The details of test specimens are summarized in Table 1.3. The influence of FRP strengthening on seismic deficient square columns was evaluated by comparing similar specimens tested under identical loading conditions. Specimens AS-1NS, AS-2NS and AS-6NS contained similarly insufficient quantities of transverse steel and subjected to an axial load equal to 33% of the nominal column capacity  $P_o$ . Column AS-1NS was the control specimen with no retrofitting, while AS-2NS and AS-6NS were retrofitted with one and two layers of CFRP, respectively. The tests showed that all ductility parameters (displacement, curvature, energy) are greater for the retrofitted columns than those for the control specimen. The second set of columns consisted of specimens AS-8NS (control), AS-4NS, AS-3NS and AS-5NS wrapped with 0, 1, 2 and 3 layers of CFRP, respectively. The specimens were similar to the first set,

except for the number of CFRP layers and the axial load level which had been increased to 56% of  $P_o$ . The test results once again showed a substantial enhancement in inelastic response of upgraded columns.

**Table 1.3** Properties of Specimens Tested by Lacobucci et al. (2002)

| Specimen | Number of CFRP layers | $P/P_o$ |
|----------|-----------------------|---------|
| AS-1NS   | 0 (control specimen)  | 0.33    |
| AS-2NS   | 1                     | 0.33    |
| AS-3NS   | 2                     | 0.56    |
| AS-4NS   | 1                     | 0.56    |
| AS-5NS   | 3                     | 0.56    |
| AS-6NS   | 2                     | 0.33    |
| AS-7NS   | 0                     | 0.33    |
| ASCR-7NS | 1 (lightly Damaged)   | 0.33    |
| AS-8NS   | 0                     | 0.56    |
| ASCR-8NS | 3 (Highly Damaged)    | 0.56    |



Typical cross section of the test specimens

The second phase of the same research program had the objective of investigating the effectiveness of CFRP in strengthening already damaged columns. Specimen AS-7NS was lightly damaged first and then repaired using one layer of CFRP and tested under an axial load of 33% of  $P_o$ , while the control specimen AS-8NS, which was tested to failure, was also repaired and rapped with 3 layers of CFRP and tested once again under an axial load of 56%  $P_o$ . The test results showed that specimen AS-7NS displayed ductility that was comparable to specimen AS-2NS. The behavior was vastly improved compared with that of unretrofitted control column AS-1NS. However, the behaviour of specimen AS-8NS after repairing clarified repair limitations for highly damaged columns. The ductility, toughness and maximum strength values were lesser than the values obtained from specimen AS-5NS.

The conclusions obtained from these research projects can be summarized below:

- The CFRP jacketing technique used substantially increased ductility and energy dissipation capacities of otherwise deficient columns.



- The effectiveness of using CFRP for repairing previously damaged columns depends on the severity of damage sustained, i.e. more CFRP layers are needed for highly degraded columns to achieve acceptable performance levels.
- Higher axial forces decrease overall column response leading to additional demands on CFRP jackets to restrain critical re-joins.
- Seismic behavior of deficient columns appropriately retrofitted with CFRP can exceed those having sufficient steel content, conforming to earthquake building standard.

***S.A. Sheikh and G. Yau (2002)***

The objective was to evaluate the effectiveness of FRP reinforcement in strengthening deficient columns and repairing damaged columns. 12 circular columns having a diameter of 356 mm and 1470 mm shear span were built and tested under constant axial compression and lateral load reversals. The range of the applied axial load ranged between 27% and 54% of section concentric capacity. The columns were divided to 3 groups as follows:

- I. First group consisted of four columns conventionally reinforced with longitudinal and spiral steel reinforcement.
- II. Second group contained six columns strengthened with CFRP or GFRP before testing.
- III. The third group included two columns that were damaged to a certain level and then repaired with FRP under axial load.

The volumetric ratio of transverse steel varied between 0.3% and 1.2%, while vertical spacing of transverse reinforcement was between 80mm and 300 mm. test results were compared against previously tested columns confined with conventional steel reinforcement.

The authors concluded the following:

- Use of CFRP and GFRP resulted in significant improvement in both ductility and strength of test columns.

- Behavior of FRP retrofitted columns under simulated earthquake loads matched or exceeded the performance of conventional steel–reinforced columns designed according to ACI-318 seismic provisions.
- FRP composites were very effective for the rehabilitation of damaged columns.

***R. Sause, K.A. Harris, S.L. Walkup, S. Pesiki, and J.M. Ricles (2004)***

The objective was to investigate the feasibility of using CFRP composite jackets in retrofitting non-ductile reinforced concrete columns. A method proposed by Seible et al (1997) to assess the ability of fiber-reinforced polymer composite jackets to achieve a desired flexural behavior is presented and verified. For this, four full-scale square building columns were tested under combined axial and lateral loads. The specimens had a 458 mm square section, reinforced with 8 – 22 mm diameter longitudinal bars and No.10 transverse ties located at 356 mm in the plastic hinge region. The amount of axial load applied was 22% of the column nominal axial capacity. Table 1.4 summarized the four test specimens.

**Table 1.4** Properties of Specimens Tested by Sause et al. (2004)

| specimen | dimension mm | Shear span mm | # of CFRP Plies    |
|----------|--------------|---------------|--------------------|
| F0       | 458x458      | 2440          | 0 (control column) |
| F1       |              |               | 6                  |
| F2       |              |               | 4                  |
| F3       |              |               | 2                  |

The results of this experimental investigation led to the following conclusions:

- Use of CFRP jackets to confine inelastic hinge regions of non-ductile square columns greatly enhances the deformation capacity of these columns without significantly increasing their strength or lateral stiffness.
- The deformation capacity increases with increasing jacket thickness (stiffness).
- The out-of-plane stiffness of CFRP jackets can be sufficient to provide lateral support to longitudinal bars under large displacement ductility demands.

- Column failure resulting from the deterioration of compression bars, as well as longitudinal bar buckling can be delayed by using jackets with greater stiffness.

***M. H. Harajli and A. A. Rteil (2004)***

A total of 12 rectangular specimens having a cross-section of 150x300mm and a shear span of 1050 mm were tested under constant axial compression and lateral deformation reversals representing seismic loading. In addition to control columns, three retrofitting strategies were applied; i) CFRP wrapping, ii) steel fiber reinforcement, and iii) minimum ordinary transverse reinforcement. All specimens were spliced with  $30d_b$  splice length as per section 12.16.1 of the ACI-318 Building Code. Two different reinforcement ratios were used, 1.5% and 3.0%. The summary of test parameters and retrofitting strategies are provided in Table 1.5.

**Table 1.5** Test parameters of specimens tested by Harajli and Rteil (2004)

| Specimen | Column reinforcement $\rho\%$ | Transverse reinforcement (within $30d_b$ ) | CFRP reinforcement                               | Volume fraction of steel fibers $V_f\%$ | Concrete strength $f'_c$ MPa |
|----------|-------------------------------|--------------------------------------------|--------------------------------------------------|-----------------------------------------|------------------------------|
| C1       | 6 # 12<br>(1.5 %)             | #8 @ 150mm                                 | –                                                | –                                       | 20.3                         |
| C1FP1    |                               | #8 @ 150mm                                 | One 300mm-wide layer                             | –                                       | 21.1                         |
| C1FP2    |                               | #8 @ 150mm                                 | One 300mm-wide layer<br>+ three 50mm-wide strips | –                                       | 21.7                         |
| C1F1     |                               | #8 @ 150mm                                 | –                                                | 1.0                                     | 21.9                         |
| C1F2     |                               | #8 @ 150mm                                 | –                                                | 2.0                                     | 21.8                         |
| C1E      |                               | #8 @ 75mm                                  | –                                                | –                                       | 21.9                         |
| C2       | 8 # 14<br>(2.7 %)             | #8 @ 150mm                                 | –                                                | –                                       | 20.3                         |
| C2FP1    |                               | #8 @ 150mm                                 | One 300mm-wide layer                             | –                                       | 21.1                         |
| C2FP2    |                               | #8 @ 150mm                                 | One 300mm-wide layer<br>+ three 50mm-wide strips | –                                       | 21.7                         |
| C2F1     |                               | #8 @ 150mm                                 | –                                                | 1.0                                     | 21.9                         |
| C2F2     |                               | #8 @ 150mm                                 | –                                                | 2.0                                     | 21.8                         |
| C2E      |                               | #8 @ 75mm                                  | –                                                | –                                       | 21.9                         |

The authors concluded the following:

- Without the confinement, the spliced reinforcement at the column base experienced significant bond deterioration under cyclic loading, leading to considerable stiffness and strength decay.
- Wrapping column critical zones with spliced reinforcement, using a relatively small area of CFRP sheets increased the bond between steel and concrete, and improved the column strength significantly while reducing concrete spalling and bond deterioration, resulting in considerable improvements in seismic performance.
- While increasing the area of FRP sheets by 50% improved the seismic performance, the improvement was found not to be proportional to the increase in FRP.
- Taking into account the modulus of elasticity of the confining material, and for the same area of confinement reinforcement per unit length along the spliced length, external confinement by CFRP is more effective in improving seismic performance than the use of conventional transverse steel.
- The level of improvement in seismic performance acquired using a given area of external or internal confinement reinforcement depended on the level of improvement of the bond behavior of the outermost column reinforcement rather than the total reinforcement ratio of the column.

***Cem Yalcin, Osman Kaya, Mustafa Sinangil (2006)***

This experimental work focused on retrofitting reinforced concrete columns using carbon fiber reinforced polymer (CFRP) sheets to improve their strength and ductility capacities. Plain rebars, inadequate splicing lengths, and insufficient stirrup spacing were the main parameters to reflect the old construction practices. This study included laboratory testing of five full-scale reinforced concrete columns of 200 mm x 400 mm x 1900 mm of dimensions (as indicated in Table 1.6). In order to represent the poor construction practice, six plain rebars of 14 mm diameter were used as longitudinal reinforcement and

plain rebars of 8 mm-diameter were used as lateral reinforcement, spaced at 300 mm on centers.

**Table 1.6** Properties of specimens, Yalcin et al (2006)

| <b>Specimen</b> | <b>f<sup>c</sup> (MPa)</b> | <b>Longitudinal reinforcement</b> | <b>Strengthening technique</b>                   |
|-----------------|----------------------------|-----------------------------------|--------------------------------------------------|
| <i>L0C0</i>     | <i>16</i>                  | <i>Continuous</i>                 | <i>None</i>                                      |
| <i>L0FRP</i>    | <i>16</i>                  | <i>Continuous</i>                 | <i>CFRP wrapping</i>                             |
| <i>L50C0</i>    | <i>17</i>                  | <i>500 mm of dowel length</i>     | <i>None</i>                                      |
| <i>L50FRP</i>   | <i>17</i>                  | <i>500 mm of dowel length</i>     | <i>CFRP wrapping</i>                             |
| <i>L50W-FRP</i> | <i>14</i>                  | <i>500 mm of dowel length</i>     | <i>Welding of the rebars +<br/>CFRP wrapping</i> |

The following conclusions were drawn from the study:

1. In columns having continuous plain rebars, L0C0 and L0FRP, the lateral load capacity of L0FRP was about 10% higher than L0C0. However, the retrofitted column with CFRP (L0FRP) exhibited dramatic increase in ductility and energy dissipation capacity.
2. In columns L50C0 and L50FRP, with lap-spliced plain rebars, the behaviour was dominated mainly by the bond failure, or slippage of longitudinal rebars. Rocking action and bar slip was visually observed during testing of L50C0 and L50FRP. Furthermore, the strain gauge readings at dowel rebars did not show any yielding, and also, the curvatures measured at 150 mm and 300 mm from the column base were different in such a way that the portion of column closer to the foundation level rotated significantly more than the upper portions, indicating that the base of column has indeed rocked. Providing a passive resistance to column L50FRP by wrapping with CFRP did not improve the behaviour significantly, resulting in almost the same lateral load and energy dissipation capacities as the unretrofitted column.
3. In the case of columns with continuous or lap-spliced reinforcement, either retrofitted or not, the column with continuous reinforcement showed about 10% more lateral load capacity.

4. In the case of L50W-FRP column specimen, the lapped rebars were welded with longitudinal rebars, and then confined by CFRP wrapping. It was apparent from the force–displacement relationship that the pinching, ductility and energy dissipation characteristics improved significantly. The strain readings on the dowel bars indicated that slip has been eliminated and the longitudinal bars did not fail at welds, and yielded at higher drift levels. The horizontal load capacity of L50W-FRP increased about 20% when compared with L50C0. Also, similar curvature measurements were obtained from 150 mm and 300 mm locations indicating that the column did not rock at the foundation level.
5. The stiffness degradation of all specimens, except for L0C0, whether retrofitted or not, showed somewhat similar results. However, at mid-range drifts, such as 2–4%, the retrofitted columns showed higher stiffness values than the control column.
6. Plain rebars caused bond failure, especially if they were not sufficiently developed, and thus, externally applied passive resistance type of retrofitting scheme did not improve the behavior any further.

***InSung Kim, James O. Jirsa, and Oguzhan Bayrak (2010)***

Different approaches for strengthening and repairing inadequate lap splices in square and rectangular reinforced concrete columns using combinations of CFRP jackets and CFRP anchors were studied. Three square columns and three rectangular columns were fabricated. The columns were rehabilitated using CFRP jackets only, CFRP anchors only, or with a combination of CFRP jackets and CFRP anchors. Both damaged and undamaged columns were strengthened and tested. The test variables were: 1) section shape and number of lap splices; 2) loading program (monotonic or cyclic); and 3) rehabilitation method (use of CFRP jackets, anchors, or both). Tests showed that after rehabilitating the columns using CFRP jackets and anchors, the failure mode changed from a brittle splice failure to yielding of column reinforcement, and the strength and deformation capacities were improved under both monotonic and cyclic loading. The findings from the column splice tests can be summarized as follows:

1. Strength and deformation capacity of a column improved more when the column was rehabilitated by a combination of CFRP jackets and anchors, relative to columns rehabilitated by CFRP jackets only or by CFRP anchors with a CFRP face sheet.
2. Rehabilitation was more effective for a square column (four or five spliced bars on a face) than for a rectangular column (10 spliced bars on a face). However, the rectangular columns exhibited good performance after rehabilitation.
3. A decrease in the number of CFRP anchors did not reduce the strength of splice if the width of CFRP material used in the anchors was maintained. The deformation capacity was improved by using more anchors.
4. The rehabilitation method using partial CFRP jackets or a CFRP sheet on one face can be applied to a column with walls. Such rehabilitation improved the deformation capacity less than columns with fully wrapped CFRP jackets. However, the same improvement in strength was achieved using partial jackets.
5. The rehabilitation using CFRP jackets and anchors was as effective as that using steel jackets and adhesive anchor bolts.

### **1.8.2 PREVIOUS RESEARCH ON FRP STAY-IN-PLACE FORMWORK FOR IMPROVED SEISMIC PERFORMANCE**

The use of FRP stay-in-place formwork for improved seismic performance of new columns is a relatively new research area that has not been fully explored. Selected study approaches in this field are presented hereafter.

#### ***F. Seible, R. Burgueno, M.G. Abdullah, and R. Nuismer (1996)***

Three 40% scale models of a circular bridge column were tested as 3.7 m high cantilever columns with a core diameter of 600mm. The objective was to investigate the behaviour of concrete filled FRP tubes under seismic loading. One specimen was a control column, with 2.66% longitudinal reinforcement ratio. The other two were cast in carbon FRP tubes of 9.5 mm thickness. The tube of one specimen was embedded inside the footing to a distance of 0.8m and had no internal steel reinforcement, whereas the other specimen had the tube set 24 mm above the footing with the reinforcement similar to the control column but only as starter bars for a length of 0.9m.

The researchers concluded that specimen with starter bars and no embedment exhibited stable hysteretic response up to 8% drift. That was similar to the control specimen. The specimen with embedded FRP tube and no starter bars matched the initial stiffness of the other two specimens before experiencing premature failure of the CFRP tube due to interlaminar shear stresses. Such failure prevented the column from absorbing the energy needed for ductile behaviour.

***N.C. Gould and T.G. Harmon, (2002)***

A total of 12 scaled cantilever columns were tested under constant axial load and cyclic shear and flexure. The objective was to verify the design expressions used to predict the behaviour of reinforced concrete columns. The columns had a circular cross-section with 180 mm diameter, and were built by filling GFRP tubes with fibres aligned in the circumference direction. The columns measured 600 mm or 1200 mm in height, having a longitudinal reinforcement ratio of 4.2% and subjected to a constant axial compression of either 133 kN or 578 kN. The concrete strength ranged between 41MPa to 55MPa. The researchers reported the following conclusions:

- Wrap strains due to shear, flexure and axial load could not be treated independently
- Columns were able to sustain wrap strains in excess of 0.004 if the wrap volume was sufficiently high.
- It was noticed that low-cycle fatigue of longitudinal reinforcement was a concern for confined columns with limited ductility.

***Z. Zhu, (2004)***

The objective of this study was to investigate the construction practicability of concrete filled FRP tubes as either cast-in-place or pre-cast members in conjunction with footing. Four specimens were tested under constant axial load of either 454kN or 909 kN and simulated lateral displacement reversals. One specimen was a control column whereas the other three were cast in GFRP tubes connected to the footing in three deferent methods as follows:

- Cast-in-place reinforced,



- Pre-cast grouted, and,
- Pre-cast post-tensioned.

All specimens had a total height of 3.12 m including the footing and the column head. The columns had a circular cross section with 310mm diameter and a concrete strength of 55MPa. The GFRP tubes had  $\pm 55^\circ$  fiber orientation with a wall thickness of 5.1mm.

The result of the experimental study led to the following outcomes:

- FRP confined columns performed much better than conventional reinforced concrete columns from strength and ductility perspectives.
- The enhanced performance was attributed to the confinement effect of the FRP tubes, which prevented spalling of concrete cover, delayed the buckling of longitudinal reinforcement and the deterioration of core concrete.
- The tube embedment further improved the performance as the tube participated effectively in the axial direction.
- The study showed that various FRP tube-footing connections presented had similar performance as long as the tubes were properly embedded or implanted into the footing. These connections were found to be feasible, constructible, and functional in terms of load transfer.

The researchers also reported that practices of the pre-cast industry could be effectively implemented for the construction of the proposed confining strategy.

#### ***Y. Shao and A. Mirmiran, (2005)***

The authors tested six specimens as simple span beam columns under constant axial load and lateral deformation reversals. The objective of this study was to investigate the behavior of concrete-filled FRP tubes under reversed cyclic loading. The tubes were fabricated using two methods, i) centrifuge casting and ii) filament winding. The centrifuge casting tubes had 12.7mm thickness, 305 mm diameter and the fibers oriented in the longitudinal direction. The filament winding tubes were 5.1 mm in thickness, 322 mm diameter and  $\pm 55^\circ$  fiber orientation. One specimen of each type of tubes had no internal reinforcement, whereas the other two were reinforced with 1.7% and 2.5% longitudinal reinforcement. The specimens were either 2.4 m or 2.7 m with concrete

strength of 27.2 MPa. The applied axial load ranged between 6% and 10% of the column concentric capacity. The researchers concluded the following:

- Thick FRP tubes with fibers aligned longitudinally showed brittle compression failure similar to over-reinforced concrete member.
- Thin FRP tubes with off-axis fibers exhibited ductile tension failure with nonlinearity resembling under-reinforced concrete member.
- The authors noted that adding internal steel reinforcement in the range of 1-2% might further improve seismic behavior of members confined with FRP tubes.

***Arash E. Zaghi , M. Saiid Saiidi , Amir Mirmiran (2010)***

The seismic performance of a concrete filled fiber reinforced polymer (FRP) tube (CFFT) bridge column was studied through shake table testing and nonlinear dynamic analyses of a one-fifth scale two-column bridge pier that also incorporated a conventional RC column. The FRP tube in the CFFT column was a prefabricated composite pipe with glass fibers aligned in  $\pm 55^\circ$  with respect to the tube axis to provide both hoop and longitudinal strengths. The column-footing interface and the shake table test set-up are illustrated in Figs. 1.16 and 1.17.



**Fig. 1.16** Embedment of the FRP tube in the footing



**Fig. 1.17** Two-column piers shake table setup

The experiment demonstrated that CFFT columns can perform as a ductile, reliable column in bridges located in seismically active regions. No visible damage was detected in the CFFT column under drift ratios as high as 7%. The tensile rupture of the FRP tube occurred during the last shake table run with 1.9 times the motion obtained during the Northridge 1994 earthquake, and under a drift ratio of 8.4%. Between the two columns, the RC column damped a larger portion of energy compared to the CFFT column; however, it experienced severe spalling of the cover concrete and extensive shear-flexure cracking. On the other hand, in the CFFT column the dissipated energy normalized by the steel content was 60% larger than that of reinforced concrete column mainly due to the spread of plasticity in the CFFT column. However, the reinforced concrete column had additional reserve energy dissipation capacity since no longitudinal bar rupture occurred. Embedment of FRP tube into the footing with a length of one and one-half times the diameter of the tube provided full fixity and the development of plastic moments. The bond-slip rotations were comparable in both columns. The experimental results demonstrated that the plastic hinge length is longer in the CFFT column, as compared to the reinforced concrete column. This results in larger spread of plasticity and reductions of local strain demands on the material.

***T. Ozbakkaloglu, and M. Saatcioglu (2004)***

Experimental investigation on the effectiveness of FRP formwork when used as transverse reinforcement for HSC columns subjected to simulated seismic loading were conducted. A total of 4 circular building columns with 270mm diameter were tested under constant axial loads of 31% to 47% of nominal concentric load capacities along with incrementally increasing inelastic deformation reversals. Properties of tested specimens are shown in table 1.7.

**Table 1.7 Properties of Circular Test Specimens**

| Column | Shear Span (mm) | D (mm) | $f'_c$ (MPa) | Layers of FRP | Longitudinal steel        |             |              | Axial load |
|--------|-----------------|--------|--------------|---------------|---------------------------|-------------|--------------|------------|
|        |                 |        |              |               | Reinforcement Arrangement | $f_y$ (MPa) | $\rho_l$ (%) | $P/P_o$    |
| RC-1   | 2000            | 270    | 90.1         | 4             | 8-no.15                   | 500         | 2.79         | 0.31       |
| RC-2   | 2000            | 270    | 75.2         | 2             | 8-no.15                   | 500         | 2.79         | 0.34       |
| RC-3   | 2000            | 270    | 49.7         | 2             | 8-no.15                   | 500         | 2.79         | 0.47       |
| RC-4   | 1200            | 270    | 75.3         | 2             | 8-no.15                   | 500         | 2.79         | 0.34       |

$\rho_l$  longitudinal reinforcement ratio

The following conclusions were drawn from the experimental study reported in this paper:

- Column tests indicate that inelastic deformability of 90 MPa concrete columns can be increased up to 12% lateral drift ratio with FRP stay-in-place formwork.
- Unlike conventional steel reinforcement that confines the core concrete FRP stay-in-place formwork provides effective confinement to the entire column section. Furthermore, the FRP casings cover the entire column surface, unlike conventional steel ties placed at certain spacing, thereby enabling full utilization of higher material strength of FRP reinforcement.
- Rupturing strain of approximately 1.5% was consistently attained during tests at column failure exceeding the 0.4% often used in the current design practice for jacketing bridge columns under low levels of axial compression
- The seismic performance of FRP encased circular columns is superior to that of columns confined by conventional spiral/hoop reinforcement. This can be attributed

to increased effectiveness of FRP casings as confinement reinforcement, as well as the characteristics of plastic hinging, which exhibits significantly more plastification and progression towards the elastic region in FRP encased columns, leading to increased hinging length.

- Column deformability decreases with increasing axial load relative to column concentric capacity ( $P/P_o$ ) and concrete compressive strength ( $f'_c$ ). It was experimentally observed that under the same level of applied axial load ( $P$ ), HSC column developed higher lateral drift capacity than a companion NSC column with the same confinement characteristics.
- Confinement models developed for steel confined concrete may result in the overestimation of flexural strength and underestimation of inelastic deformability of FRP confined columns.
- FRP casings develop transverse tensile stresses caused by the combined action of shear and flexure. The contribution of shear increases under high levels of shear stress reversals, as well as following concrete deterioration in plastic hinge regions. The concrete shear resistance may diminish very significantly in the latter case, increasing the strain demand on FRP casing.

***T. Ozbakkaloglu, and M. Saatcioglu (2004)***

Experimental investigation on the effectiveness of FRP formwork when used as transverse reinforcement for HSC columns subjected to simulated seismic loading were conducted. A total of 6 square building columns with 270mm dimension were tested under constant axial loads of 31% to 47% of nominal concentric load capacities along with incrementally increasing inelastic deformation reversals. Properties of tested specimens are shown in table 1.8.

In this study, a new type of stay-in-place formwork was developed and verified in the experimental program, consisting of exterior FRP casings and internal FRP crossties. Crossties are typically required in conventionally reinforced columns and they improve the distribution of lateral restraints against concrete expansion, improving the uniformity of confinement pressure. A similar improvement is anticipated in columns confined with

FRP casings if internal crossties can be provided. Either one or two crossties in each cross-sectional direction were placed inside the casing, emulating 8-bar and 12- bar arrangements with crossties while providing clamping forces to the exterior shell. This was investigated as part of the test parameters.

**Table 1.8 Properties of Square Test Specimens**

| Column | Shear Span (mm) | D (mm) | $f'_c$ (MPa) | Layers of FRP | Longitudinal steel        |             |              | Axial load |
|--------|-----------------|--------|--------------|---------------|---------------------------|-------------|--------------|------------|
|        |                 |        |              |               | Reinforcement Arrangement | $f_y$ (MPa) | $\rho_l$ (%) | $P/P_o$    |
| RS-1   | 2000            | 270    | 90.1         | 5             | 4-No.20                   | 476         | 1.66         | 0.32       |
| RS-2   | 2000            | 270    | 90.1         | 5             | 8-No.15                   | 500         | 2.24         | 0.31       |
| RS-3   | 2000            | 270    | 90.1         | 5             | 12-No.15                  | 500         | 3.36         | 0.30       |
| RS-4   | 2000            | 270    | 75.2         | 3             | 8-no.15                   | 500         | 2.24         | 0.34       |
| RS-5   | 2000            | 270    | 75.2         | 2             | 8-No.15                   | 500         | 2.24         | 0.34       |
| RS-6   | 2000            | 270    | 75.2         | 3             | 8-No.15                   | 500         | 2.20         | 0.34       |

$\rho_l$  longitudinal reinforcement ratio

The following conclusions were drawn from the experimental investigation reported in this paper:

- High-strength concrete columns confined by carbon FRP stay-in-place formwork can develop ductile behavior under simulated seismic loading. The use of FRP formwork as confinement reinforcement substantially increases deformability of square columns. Column tests indicate that inelastic deformability of 90 MPa concrete square columns can be increased up to 11% lateral drift ratio with FRP stay-in-place formwork.
- The increased confinement requirements for HSC columns can be met by using FRP stay-in-place formwork. Unlike conventional steel reinforcement that only confines the core concrete, FRP stay-in-place formwork confines the entire column. Furthermore, unlike the discrete nature of conventional steel reinforcement, FRP formwork provides continuous confinement, covering the entire column face, resulting in higher confinement efficiency.

- The strain data recorded during column tests indicate that the strength of FRP stay-in-place formwork could be mobilized to a large extent, developing strains of up to 1.5%, approaching the tensile capacity of material. The observed strains were consistently higher than the 0.4% used in current design recommendations as a safe design limit.
- The ratio of corner radius to column dimension ( $R/D$ ) has a significant influence on the effectiveness of FRP as confinement reinforcement. Increased corner radius promotes effectiveness of FRP, while preventing premature material failure associated with sharp corners. Columns with three plies of FRP, tested in the experimental program, were able to develop 6% drift ratio prior to significant strength decay when the  $R/D$  ratio was  $1/6$ , whereas the column with sharper corners ( $R/D = 1/34$ ) was able to develop a limited drift ratio of about 2%.
- The use of FRP crossties improves the efficiency of FRP stay-in-place formwork, in much the same manner as overlapping hoops and crossties used in conventional steel confinement reinforcement. Therefore, the concept of integrated crossties in FRP casings, introduced by the authors, has proven to be effective.
- Flexural capacity of columns with FRP stay-in-place formwork can be computed using the plane section analysis commonly employed for reinforced concrete members, provided that appropriate material models are incorporated.

***T. Ozbakkaloglu, J.C. Lim and T. Vincent (2012)***

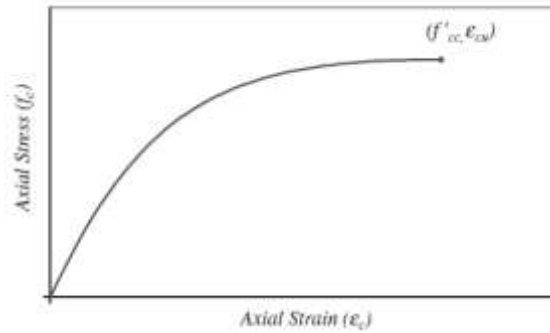
A comprehensive assessment of existing models to predict the axial stress-strain behaviour of FRP-confined concrete in circular columns has been reviewed. A total of 88 models were summarized and classified into two categories, namely design oriented and analysis oriented models. Of the reviewed models, 59 were design-oriented and 29 were analysis-oriented. Test database of FRP-confined concrete cylinders tested under monotonic axial compression is used for model assessment. The database covered 2038 test results from 202 experimental studies and sorted based on two main confinement parameters; that is, confinement technique (wraps or tubes) and type of FRP material (Carbon FRP (CFRP), S- or E-Glass FRP (GFRP), Aramid FRP (AFRP), or High-modulus Carbon FRP (HM CFRP)). The model performances are evaluated based on

their predictions of the strength enhancement ratios ( $f'_{cc}/f'_{co}$ ) and strain enhancement ratios ( $\epsilon_{cu}/\epsilon_{co}$ ).

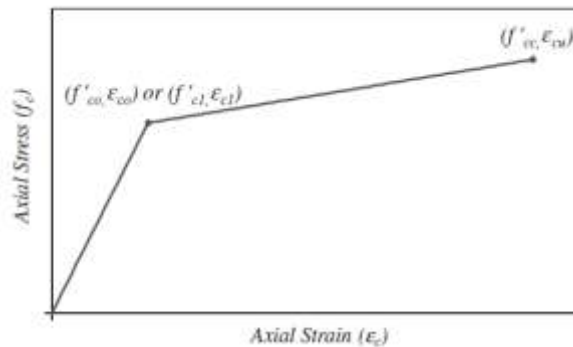
Model performances were quantified using four statistical indicators: the mean square error (MSE), the average absolute error (AAE), the linear trend slope (LTS) and the standard deviation (SD). The reliability of model predictions depends on the size and completeness of the database used in model development. Through this study, references have been made to models developed for steel-confined concrete as it played vital role in developing the FRP models.

### Design oriented models

Design-oriented models are classified into three categories based on the geometric form of the curves (i.e. Type I for parabolic, Type II for bilinear, and Type III for combination of both). Type III curves are further divided into three sub-categories according to the approach they adopted in the development of their proposed curves, namely as Type IIIa, IIIb and IIIc, as shown on Fig 1.18 to 1.20.

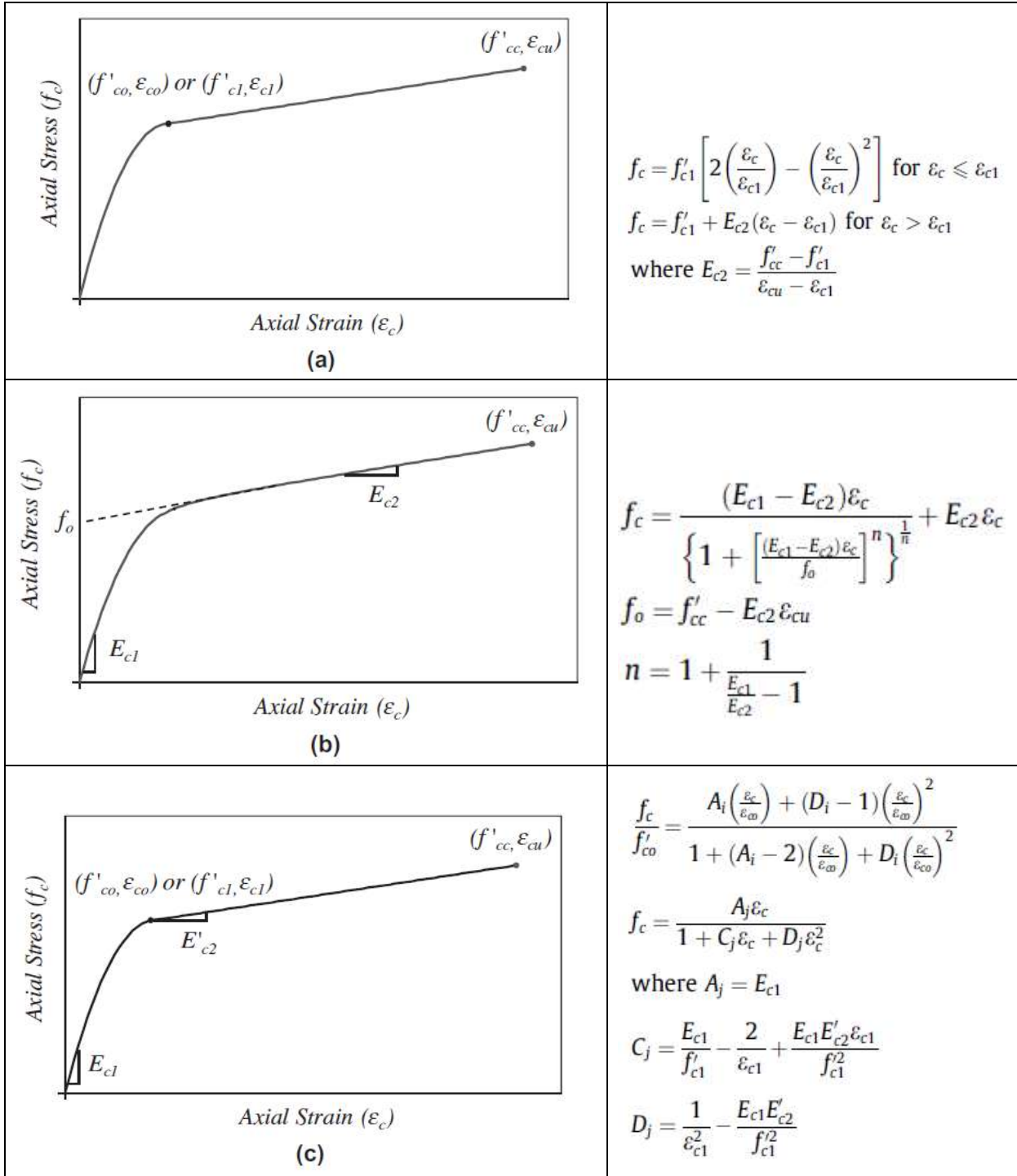


**Fig 1.18** Stress–strain curve of FRP-confined concrete based on steel or actively confined concrete models – Type I



**Fig 1.19** Bilinear stress–strain curve of FRP-confined concrete – Type II

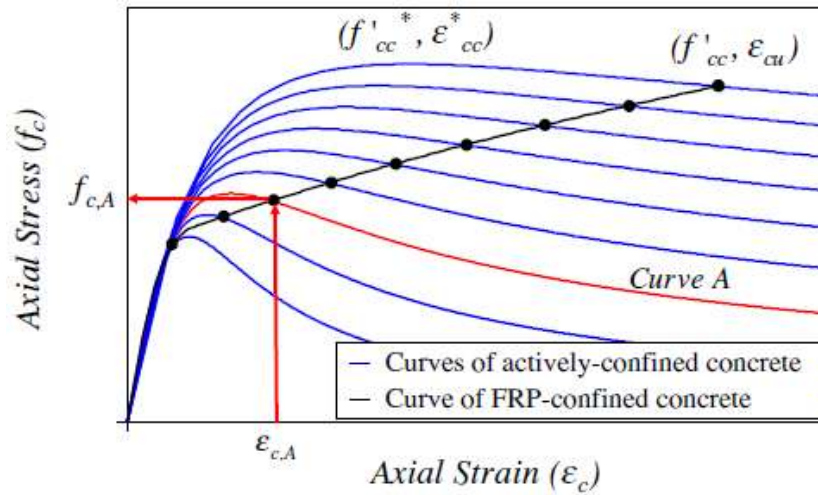




**Fig.1.20** Stress–strain curves of FRP-confined concrete with parabolic initial ascending branch and linear or quasi-linear second branch: (a) type IIIa; (b) type IIIb; (c) type IIIc.

### Analysis oriented models

In the development of analysis-oriented models for FRP-confined concrete, the stress–strain curves of actively confined concrete are employed as the base curves. The axial stress– strain curve of FRP-confined concrete is then obtained through an incremental approach, with the resulting curve crossing a family of stress–strain curves for the same concrete under different levels of active confinement pressure (Fig. 1.21).



**Fig.1.21** Determination of stress-strain curves of FRP-confined concrete by analysis-oriented model

The following observations and conclusions were drawn from this study.

1. Design-oriented models perform better than analysis-oriented models. Furthermore, performance of design-oriented models increase with the size of database used in their development
2. Design- oriented models by Lam and Teng and by Tamuzes at al. are the most accurate for the prediction of ultimate strength and strain enhancement ratios
3. The use of relatively small test database compromises model accuracy
4. Accuracy of models increases when using hoop rupture strains rather than ultimate tensile strains of fiber
5. Average values of hoop strain reduction factor based FRP composites are determined as 0.685

6. Prediction errors associated with ultimate axial strains  $\varepsilon_{cu}$  are larger than those of ultimate strengths  $f'_{cu}$ .

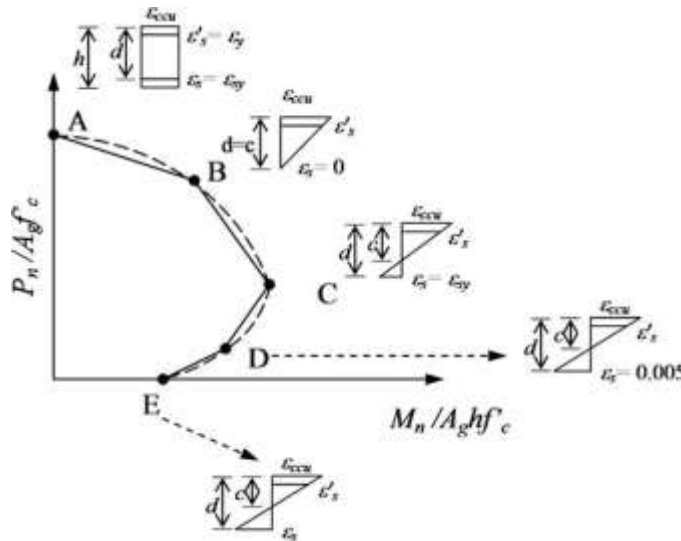
**Silvia Rocca, Nestore Galati and Antonio Nanni (2007)**

In this paper, a design method to construct a simplified interaction diagram for FRP-wrapped columns. The scope consists of 4 steps;

- Collection of experimental data on FRP-wrapped circular and non-circular columns with FRP fiber orientation perpendicular to the longitudinal axis of the column
- Construction of P-M diagram based on principle of equilibrium and strain compatibility considering a stress-strain model for FRP-confined concrete
- Validation of the theoretical P-M diagram with experimental results
- A simplification of the P-M diagram to a series of straight lines in the compression region of the diagram corresponding to 5 characteristic points as shown in the figure.

For this, experimental data from 13 circular and 48 non-circular RC columns subjected to a combination of axial compressive load and flexure were collected.

#### **P–M interaction diagram**



**Fig. 1.22** Simplified interaction diagram for circular and non-circular columns

The analysis of the FRP-confined specimens is equivalent to that of customary RC columns with the primary difference being the use of a selected stress–strain model for FRP-confined concrete in the compression zone. The following assumptions are considered for the analysis: (a) plane sections remain plane, (b) the tensile strength of concrete is neglected, and (c) complete composite action is assumed between both steel reinforcement-concrete and FRP-concrete.

P–M diagram was constructed as a series of straight lines joining the axial load and moment values corresponding to five characteristic points as illustrated in Fig. 1.22. The proposed design method was based on ACI protocols and limiting values. In the interaction diagram, point A represents the pure compression case (zero bending moment). For points B, C, and D, the position of the neutral axis  $c$  is directly computed by similar triangles in the strain distribution corresponding to each case. Point E, as representing the pure bending moment case (zero axial force), the neutral axis position is obtained by following conventional RC beam theory.

The obtained values were plotted against experimental data at the same scale. All plots are drawn with x-axis equal to  $M_n / (A_g h f'_c)$  and y-axis equal to  $P_n / (A_g f'_c)$ . Comprehensive calculation procedure for the proposed design methodology was presented accompanied by a complete example application for clarity. The paper concluded that plotted data points appear to be consistent with the analytical predictions following the principles of ACI building code.

# **CHAPTER 2**

## **EXPERIMENTAL RESEARCH**

### **SPECIMEN DETAILS AND MATERIAL TESTING**

#### **2.1 GENERAL**

The experimental program was designed to investigate the effectiveness of using CFRP composites as transverse reinforcement for retrofitting existing bridge columns and for new construction as FRP stay-in-place formwork. It constitutes an important component of the current research program, providing much needed data to develop design and construction techniques for earthquake resistant concrete columns. A total of seven full-scale concrete bridge columns were designed, built and tested under simulated seismic loading. Four specimens represented existing non-ductile bridge columns, retrofitted with CFRP wrapping. The remaining three columns were used to develop a new FRP stay-in-place formwork for use in new construction. CFRP casings, having either a circular or a square geometry, with and without internal crossties were manufactured as column formwork. The formwork remained on the columns and provided transverse reinforcement for improved deformability, while also providing durability against corrosion. It is noteworthy that, unlike building columns, bridge columns are subjected to low levels of axial compression, which typically range between 10% to 20% of column concentric load capacity. By looking at the interaction diagrams generated for proposed test specimens, these loads are significantly below the balanced axial load as shown in Fig.2.1, and typical failure mode in such case is dominated by the yielding of steel reinforcement rather than concrete crushing. The proposed retrofitting technique is aiming to providing adequate lateral confinement to insure satisfactory inelastic behavior of the plastic hinge and to prevent the longitudinal bars from buckling prematurely or lap spliced from debonding. This Chapter provides the details of the experimental program

in terms of test setup, specimen preparation, instrumentation, test procedure and materials testing. Test results have been evaluated and presented in Chapter 3.

## **2.2 CONSTRUCTION OF TEST SPECIMENS**

Test columns were constructed at the Structures Laboratory of the University of Ottawa. The material, including ready-mix concrete and bent reinforcement was acquired from outside. The assemblies of column cages and concrete casting, as well as the production of FRP composites, were all performed at the Structures Laboratory. The specimens represented circular or square reinforced concrete bridge columns, either between the fully fixed foundation and a hinge support at the superstructure level or a portion of bridge columns with fixed ends, between the fully fixed foundation and the point of inflection.

### **2.2.1 FOOTINGS**

Each column was produced with its footing. The footings were well reinforced to ensure full fixity and to prevent any premature failure during testing. Figs. 2.2 show the footing geometry, reinforcement details and formwork preparation. Precautions were taken to keep the starter or longitudinal column bars in place during casting, while the strain gauges placed on re-bars were well sealed against possible damage during casting.

### **2.2.2 COLUMNS**

The construction of specimens was done in two stages to simulate the actual practice in the field. First the footings were cast, and then the column cages were built. Three different types of formwork were used to cast the column concrete; i) Sona tubes for casting circular columns BR-C8-2, BR-C10-2 and BR-C6-1, ii) square plywood formwork built in the laboratory for column BR-SS-R, and iii) CFRP stay-in-place formworks that were manufactured in the laboratory for columns BR-C6-1, BR-SC-1 and BR-SC-2. Figs. 2.3 and 2.4 illustrate plywood and CFRP formworks used for casting. The columns were wrapped with Hessian cloth and polyethylene sheets and cured for one week to ensure proper strength gain.

### 2.2.3 CFRP COMPOSITE SYSTEM

The advanced composite system used to produce CFRP retrofitting and formwork was manufactured from **Wabo Mbrace** carbon fiber sheets produced by **Watson Bowman Acme**. The sheets consist of high performance dry fabrics, manufactured to have very high strength aerospace grade carbon fibers. They are impregnated in polymer resins to form a composite material. Three different epoxy based materials were used as explained below:

- **Primer:** a low viscosity epoxy material that penetrates the pore structure of the substrate. Its function is to insure proper bond on the substrate.
- **Putty:** A thick epoxy paste used to fill in and smooth small surface defects and irregularities on concrete surface.
- **Saturant:** A sag-resistant epoxy impregnation resin, used to wet out and encapsulate carbon fiber fabrics forming a high performance FRP laminate.

#### 2.2.3.1 CFRP SYSTEM INSTALLATION FOR COLUMN RETROFITTING

This fabric-based system is installed by a technique known as wet lay-up. It involves applying the lightweight, flexible fiber fabrics onto a prepared surface of the column under consideration, using the uncured epoxy resin (Saturant). Once the resins cure, the result is a high-strength bonded FRP laminate that is comparable to bonding steel plates. However, this system is easier to install compared to steel plates, needs minimal labor and can be applied to areas of difficult access. This system also acts as a protection for the underlying structure against harsh environmental effects. The composite system along with the hardware used for installation is illustrated in Fig 2.5. Step-by-step method of application of the FRP fabric on the concrete specimens is shown in Fig 2.6 and 2.7. Fig 2.8 shows completed column wrapping.

#### 2.2.3.2 PREFABRICATED CFRP CASINGS

Apart from the standard fabric application explained above, CFRP sheets can be used for new construction in the form of rigid CFRP molds or Stay-in-Place formwork, a

relatively new concept that is currently being explored. The intent was to benefit from multiple advantages that this material offers during new construction. These molds (also referred to as casing), once built, can be filled with fresh concrete and internal longitudinal reinforcement, while staying on the column as a permanent FRP jacket to increase ductility and to protect the structural member against environmental attacks. Accordingly, one circular and two square molds were designed and built in the Structures Laboratory. The circular FRP mold was built using a Sonatube as a template, while a wooden template was used as a guide to manufacture the square casings. Three layers of CFRP fabric were used to form the circular casing and four plies to form the square ones. Two different types of square tubes were manufactured, i) a square tube with single CFRP crossties embedded in each direction and ii) a square tube with double CFRP crossties embedded in each direction to emulate column crossties. Crossties from the same CFRP fabric were manufactured separately and then integrated with the square casing. Images for the circular casing along with a photo of column BR-CC-1 that utilized this type of casing are shown in Fig 2.9. The square casing used for column BR-SC-1 is shown in Fig 2.10. The figure also illustrates the crosstie configuration within the casing.

#### 2.2.3.3 DETAILS OF CFRP CROSSTIES

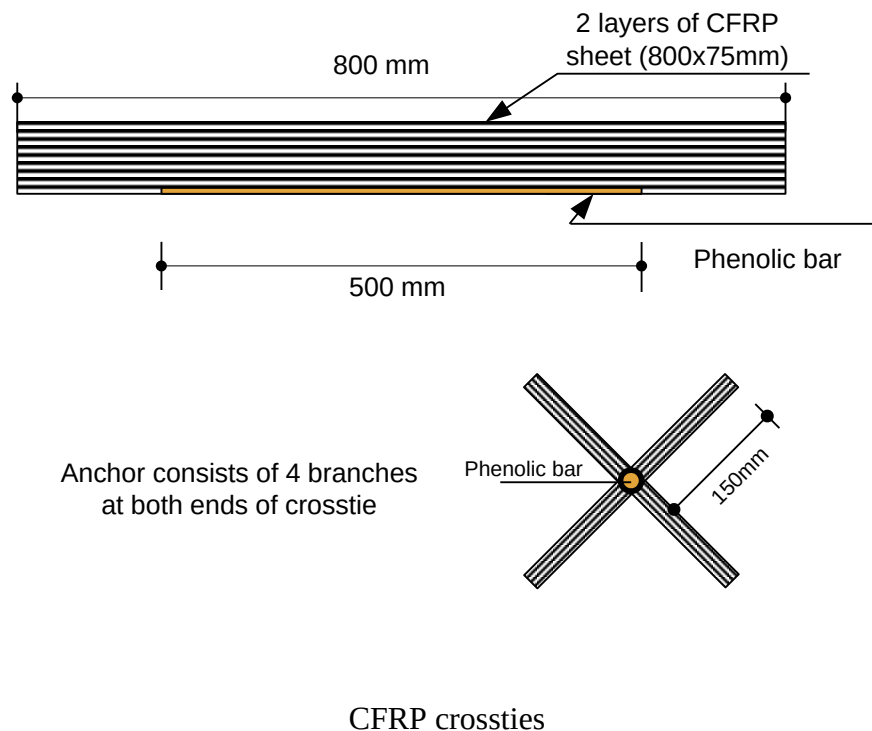
Cross ties are typically provided in conventionally reinforced concrete columns to improve the distribution of lateral restraints against concrete expansion and the uniformity of confinement pressure. The same benefits can be anticipated if internal crossties can be provided in columns confined with CFRP casing. Accordingly, crossties made from the same material (i.e. CFRP) were provided to square CFRP casings used in columns BR-SC-1 and BR-SC-2. They had been produced by rapping resin-impregnated straight-cut pieces of the fiber sheet around low tensile-strength phenolic bars (of cellulose base) to form rounded CFRP rods. The phenolic bars were used as a template and didn't contribute to the strength of the crossties.

The length of the template bars was equal to the column width (500mm) while the carbon sheet extended by 150mm beyond both ends to allow for integration process. As the



casing consisted of 4 layers, small holes with 12 mm diameter were drilled after the completion of the second layer to insert the crossties. Once inserted, the fibers at the ends were bent and glued to the casing surface using epoxy resin. Following the completion of crosstie insertion, the remaining two layers of FRP sheets were applied to form a complete casing. Fig 2.11 shows the crosstie preparation process.

The fiber content used to produce one crosstie was governed by the size of the FRP sheet cut. Each crosstie contained fibers equal to the area of fibers placed on one face of a column within the spacing of crossties. As the crossties spacing was 150 mm in the vertical direction, the fiber area of each crosstie was equal to  $500 \times 150 = 75000 \text{ mm}^2$ . In column BR-SC-2 double crossties were used in each direction, the amount of fibers placed in FRP crossties was equal to one ply of CFRP casing. A top close-up view of crossties integrated within the casing, as well as the strain gauge wiring is illustrated in Fig 2.12a and b. The following sketch illustrates the FRP sheets used for producing the internal FRP crossties.



## **2.3 SPECIMENS DETAILS**

The following is a description of the seven reinforced concrete column specimens prepared for the current research program.

### **2.3.1 LONG CIRCULAR COLUMN BR-C8-2**

The specimen BR-C8-2 was 1.7 m in height and had a shear span of 2.0 m, measured to the centerline of the horizontal actuator that was positioned 275 mm above the tip of the specimen. The shear span was selected to generate flexure dominant behavior. The column had a 500 mm circular cross-section. A total of 12 –No.20 bars were placed as longitudinal flexural reinforcement with a 50 mm clear concrete cover. The longitudinal reinforcement was spliced near the base; within the potential plastic hinge region with a lap splice length of 20-bar diameter, representing the pre-1971 practice. Figure 2.15 shows the column geometry, the reinforcement detailing, and the steel cage prior to casting. The retrofitting is carried out by wrapping the column with 2 plies of CFRP fabric (Fig 2.8).

### **2.3.2 SHORT CIRCULAR COLUMN BR-C10-2**

Specimen BR-C10-2 had a circular cross section with a diameter of 610 mm. It was reinforced with 12-No.20 bars with a clear cover of 50 mm. The column had a reduced height of 1.2 m with a corresponding shear span of 1.5 m, measured to the center of application of the horizontal force. The column geometry and reinforcement detailing are shown in Fig 2.16. The specimen was retrofitted with 2 plies of CFRP fabric (Fig 2.7).

### **2.3.3 LONG CIRCULAR COLUMN BR-C6-1**

The column was companion to BR-C8-2 with similar properties, except that the longitudinal reinforcement was continuous, without the lap splices. The geometry and reinforcement for specimen BR-C6-1 are illustrated in Fig 2.17. It was retrofitted by wrapping 2 plies of CFRP fabric. The retrofitted specimen is shown in Fig 2.8.

#### **2.3.4 SHORT SQUARE COLUMN BR-SS-R**

This is a 500x500 mm square column with 1.2 m height and 1.5 m shear span for shear dominant behavior. The column was reinforced with 12-No.20 continuous longitudinal reinforcement, without lap splicing. All the details, including dimensions, reinforcement and steel cage prior to casting, are shown in Fig. 2.19. The column was retrofitted by wrapping with 3 plies of CFRP fabric, as illustrated in Fig 2.8.

#### **2.3.5 LONG CIRCULAR COLUMN BR-CC-1**

Specimen BR-CC-1 was a circular column with 508 mm diameter and 2.0 m shear span. It was reinforced with 12-No.20 spliced longitudinal reinforcement with a lap length of 20 times the bar diameter. The column geometry and reinforcement details were similar to those of BR-C8-2 as shown in Fig 2.18. The column did not contain any transverse reinforcement other than the CFRP casing used as stay-in-place formwork, also providing transverse confinement reinforcement. After the placement of longitudinal reinforcement in the CFRP formwork, concrete was cast. The prefabricated CFRP casing and the specimen after casting are illustrated in Fig 2.9.

#### **2.3.6 LONG SQUARE COLUMN BR-SC-1**

Column BR-SC-1 was companion to BR-CC-1, except it had a square cross-section. The column geometry and reinforcement details are illustrated in Fig 2.20. The column was cast in a prefabricated square CFRP casing measured 500x500 mm in cross section. The casing was manufactured to contain integrated cross ties in both directions, placed in the middle of each side, with the first tie 50 mm from the column-footing interface followed by a constant spacing of 150 mm along the column height, as depicted in Fig 2.10. The reinforcement arrangement differed in this specimen compared to the others. A total of 8 #25 bars were used instead of the 12 #20 bars used in other columns to allow for the placement of a central bar in the corner of each crosstie and the perimeter casing. The purpose of providing internal crossties was to investigate the ability of ties to constrain the lateral deformation of casing to improve the mechanism of confinement, as well as to

support the vertical bars against buckling. Top view of the column cross-section, showing the ties and the reinforcement, is illustrated in Fig 2.12b.

### **2.3.7 LONG SQUARE COLUMN BR-SC-2**

Column BR-SC-2 was manufactured similar to BR-SC-1, using a 500 x 500 mm square prefabricated CFRP casing with two CFRP cross ties at each tie level in both cross-sectional directions, and arranged vertically with 150 mm spacing. The area of carbon sheets used to produce one crosstie was equal to the area of one side of the casing within a tie spacing i.e.  $(500 \times 150) \text{ mm}^2$ . The cross ties were intended to replace the conventional steel ties as transverse reinforcement. The longitudinal reinforcement consisted of 12 #20 bars, one bar placed in the corner of each crosstie and the perimeter casing. The longitudinal reinforcement was lap spliced, following the current splice requirements for earthquake resistant columns. This resulted in 60 times the bar diameter, or 1200 mm. The column geometry and reinforcement details are illustrated in Fig 2.21. Figure 2.13 illustrates the steps followed during the manufacturing process for which a wooden skeleton was used as a guide to wrap the sheets (Fig. 2.13a). Two layers were wrapped first, prior to the insertion and gluing of cross ties as shown in Fig. 2.13b. Fig. 2.13c shows the CFRP ties in place. The completed casing is illustrated in Fig. 2.13d and consisted of a total of 4 CFRP layers. The steel arrangement relative to crossties is shown in Fig. 2.14c. The middle bars were placed adjacent to the crossties to investigate their ability to support the bars against buckling. The column cage is shown in Fig. 2.14d just before the placement of concrete.

Tables 2.1 and 2.2 summarize the specimen details. For columns with spliced bars, the lap splice length was 20 bar diameter, except for column BR-SC-2, which had a spliced length of 60 bar diameter.

## 2.4 MATERIAL PROPERTIES

The properties of materials used in constructing the columns were established through standard tests. The following sections provide the results of concrete cylinder tests, as well as coupon tests for steel and FRP.

**Table 2.1** Column Geometry and Reinforcement

| COLUMN*  | DIMENSION (mm) | SHEAR SPAN (m) | LONGITUDINAL REINFORCEMENT     | TRANSVERSE REINFORCEMENT |
|----------|----------------|----------------|--------------------------------|--------------------------|
| BR-C8-2  | 508, Diameter  | 2.0            | 12 –No.20 spliced ( $20 d_b$ ) | No.10 @300 mm            |
| BR-C10-2 | 610, Diameter  | 1.5            | 12 –No.20 spliced ( $20 d_b$ ) | No.10 @300 mm            |
| BR-C6-1  | 508, Diameter  | 2.0            | 12 –No.20 continuous           | No.10 @300 mm            |
| BR-CC-1  | 508, Diameter  | 2.0            | 12 –No.20 spliced ( $20 d_b$ ) | Not provided             |
| BR-SS-R  | 500x500        | 1.5            | 12 –No.20 continuous           | No.10 @300 mm            |
| BR-SC-1  | 500x500        | 2.0            | 8 –No.25 spliced ( $20 d_b$ )  | 1 CFRP crosstie @150mm   |
| BR-SC-2  | 500x500        | 2.0            | 12 –No.20 spliced ( $60 d_b$ ) | 2 CFRP crossties@150mm   |

\* BR is for Bridge Column, C for Circular and/or Casing, S for Square and/or Short, and R for Wrapped.

**Table 2.2** Column Retrofit/Casing Details

| COLUMN   |       |               | RETROFITTING/CASING                     |
|----------|-------|---------------|-----------------------------------------|
| LABEL    | TYPE  | CROSS-SECTION |                                         |
| BR-C8-2  | Long  | Circular      | Retrofitted, 2 plies                    |
| BR-C10-2 | Short | Circular      | Retrofitted, 2 plies                    |
| BR-C6-1  | Long  | Circular      | Retrofitted, 2 plies                    |
| BR-CC-1  | Long  | Circular      | CFRP Casing, 3 plies                    |
| BR-SS-R  | Short | Square        | Retrofitted, 3 plies                    |
| BR-SC-1  | Long  | Square        | CFRP Casing, 4 plies (single crosstie)  |
| BR-SC-2  | Long  | Square        | CFRP Casing, 4 plies (double crossties) |

### 2.4.1 CONCRETE

Concrete used for casting the footings and the columns were supplied by a local ready-mix company. Sufficient number of standard cylinders were cast and cured with the specimens to monitor the strength gain, as well as to establish the stress-strain relationships. A Fourney testing machine with 2225 kN capacity was used to test the

cylinders as illustrated in Fig. 2.22. Three different batches of concrete were used to cast the columns. The first set of columns cast consisted of BR-C8-2, BR-C10-2 and BR-C6-1 and the 28-day concrete strength for these columns was 38 MPa. Second set included specimens BR-CC-1, BR-SC-1 and BR-SS-R and the strength obtained for this set was 33MPa. Column BR-SC-2 was the last to be cast, and the concrete strength was 34 MPa.

#### 2.4.2 REINFORCING STEEL

Coupon tests for the reinforcing steel were performed on **Galdabini SUN60** testing machine as shown in Fig. 2.23. This figure also shows a coupon test set-up and the reinforcement after testing. Three different sizes of bars were used; No. 10 bars with a nominal diameter of 11.3 mm, No. 20 bars with a nominal diameter of 19.5 mm and No. 25 mm bars with a nominal diameter of 25.2 mm. The stress-strain relationship (Fig. 2.24) was established by performing three coupon tests. Table 2.3 provides average properties of each type of reinforcement obtained.

**Table 2.3** Properties of the reinforcing steel

|        | $E_s, MPa$ | $f_y, MPa$ | $\epsilon_y$ | $\epsilon_{sh}$ | $f_u, MPa$ | $\epsilon_u$ |
|--------|------------|------------|--------------|-----------------|------------|--------------|
| No. 10 | 178,800    | 447        | 0.0025       | 0.0093          | 681        | 0.117        |
| No. 20 | 191,666    | 460        | 0.0024       | 0.0069          | 716        | 0.130        |
| No. 25 | 200,909    | 442        | 0.0022       | 0.0103          | 682        | 0.131        |

#### 2.4.3 CFRP COMPOSITE

Carbon fiber polymer composite material was used to wrap the specimens, as well as to manufacture the stay-in-place formwork. The thickness of the carbon fiber sheet as provided by the manufacturer was 0.165 mm/ply, which correspond to 0.8 mm/ply when impregnated with epoxy resin. The fibers were oriented in the circumferential direction to develop hoop tension. The strength, elastic modulus and the stress-strain relationship of CFRP composites were established through coupon tests, showing linear elastic behavior up to the rupturing strength in tension. The coupons were taken from actual column wrapping after the column tests were completed. They were prepared following the

method outlined in the ASTM standards D3039 – 76 (Re-approved 1982). Fig. 2.25 depicts the coupons. The graph showing the test results is illustrated in Fig. 2.26. The tests showed that the coupons failed in tension at average tensile stress of 780MPa and tensile strain of 1.2%, based on the fiber content corresponding to 0.8mm/ply. This observation was found to be in agreement with the 3800 MPa fiber strength reported by the manufacturer. The tensile strength for the two cases is plotted in Fig. 2.27.

## 2.5 TEST SETUP

The test setup consisted of five components; i) two vertical actuators for the application of gravity loads, ii) a horizontal actuator to apply incrementally increasing deformation reversals, iii) A-frame assembly that provided lateral support, iv) a loading beam that was attached to the column for the application of gravity and lateral loads, and v) two steel frames to restrain the specimens against large out-of-plane movements. Figure 2.28 show the overall view of the test setup. Side elevation is shown in figure 2.29.

The test columns were attached to the laboratory strong floor with four high-strength bolts of 75 mm diameter to ensure full fixity during testing. The three servo-computer controlled MTS hydraulic actuators had a 508 mm-stroke and could apply a load of 1000 kN in tension and 1500 kN in compression. The schematic drawing shown in Fig. 2.29 illustrates the main characteristics of the actuators. There was one actuator on each side of the column (see Fig. 2.30) to apply uniform axial compression, simulating gravity loading. Each vertical actuator was attached to the footing and to the loading beam assembly. The third actuator was mounted horizontally on the A-frame that was bolted to the strong floor (Fig. 2.29). One end of the actuator was attached to the A-frame and the other end to the tip of the column. The horizontal actuator applied a reversed cyclic loading in displacement control mode on the tip of the specimen to simulate seismic loading. Four high-strength bolts were used to connect the actuators. The entire setup was symmetrical to avoid eccentric loading (see Figs. 2.30 and 2.32). Schematic drawings present details of the test setup in Figs. 2.30 through 2.33.

## 2.6 LOADING PROGRAM

A constant axial compressive load was applied on each specimen during testing. This load represented gravity loads of the superstructure. Bridge columns are usually subjected to a gravity load of 10% to 20% of their axial capacity under pure compression  $P_o$  (Yalcin, 1998). Axial load level, corresponding to 15%  $P_o$  was computed and applied on the columns where  $P_o$  was computed using Eq. 1.

$$P_o = 0.85 f'_c (A_g - A_s) + A_s f_y \quad (1)$$

The compressive load applied on each specimen is listed in Table 2.4.

**Table 2.4** Compressive load applied to test specimens

| COLUMN   |                                         | LOAD APPLIED |
|----------|-----------------------------------------|--------------|
| LABEL    | DESCRIPTION                             | 15% $P_o$    |
| BR-C8-2  | Long circular column (spliced)          | 1180 kN      |
| BR-C10-2 | Short circular column (spliced)         | 1614 kN      |
| BR-C6-1  | Long circular column (continuous)       | 1180 kN      |
| BR-CC-1  | Long circular column (spliced – casing) | 1000 kN      |
| BR-SS-R  | Short square column (continuous)        | 1200 kN      |
| BR-SC-1  | Long square column (spliced – casing)   | 1220 kN      |
| BR-SC-2  | Long square column (spliced – casing)   | 1230 kN      |

The horizontal cyclic loading was applied in displacement-control mode. The loading regime consisted of three complete lateral load reversals (cycles) applied at each drift ratio. Each specimen was cycled at 0.5% drift, with the subsequent cycles at 1%, 2%, 3%, 4%, etc. until the specimen became unable to sustain significant load. The loading rate was slow and depending on the amount of ductility each column was able to sustain.



Typical test duration lasted between four to five hours. The loading program is illustrated in Fig. 2.34.

## 2.7 INSTRUMENTATION

The specimens were well instrumented. The data was recorded using a data acquisition system. Four types of instruments were used to measure various quantities:

- Calibrated load cells of actuators (see Fig. 2.30).
- Electrical-resistance strain gauges bonded on reinforcing bars and CFRP sheets
- Linear Variable Differential Transducers (LVDT) of 50 mm-stroke
- Temposonic LVDT's for displacement measurements.

The strain gauges were placed on longitudinal and transverse reinforcement. Additional strain gauges were placed on CFRP jackets/casings, as well as CFRP crossties. The locations of strain gauges for each specimen are illustrated in Chapter 3, along with recorded values.

A total of four LVDT's were used to record flexural rotations at  $h$  and  $h/2$  distances away from the footing, where  $h$  is the cross-sectional dimension of columns. They were secured on threaded rods, which had been cast into concrete columns. The following equation (Eq.2) was used to calculate flexural rotations  $\theta_h$  or  $\theta_{h/2}$  from recorded displacements.

$$\theta = \text{Arc tan} \left( \frac{\Delta_1 - \Delta_2}{d_{LVDT}} \right) \cong \left( \frac{\Delta_1 - \Delta_2}{d_{LVDT}} \right) \quad \text{for small angles} \quad (2)$$

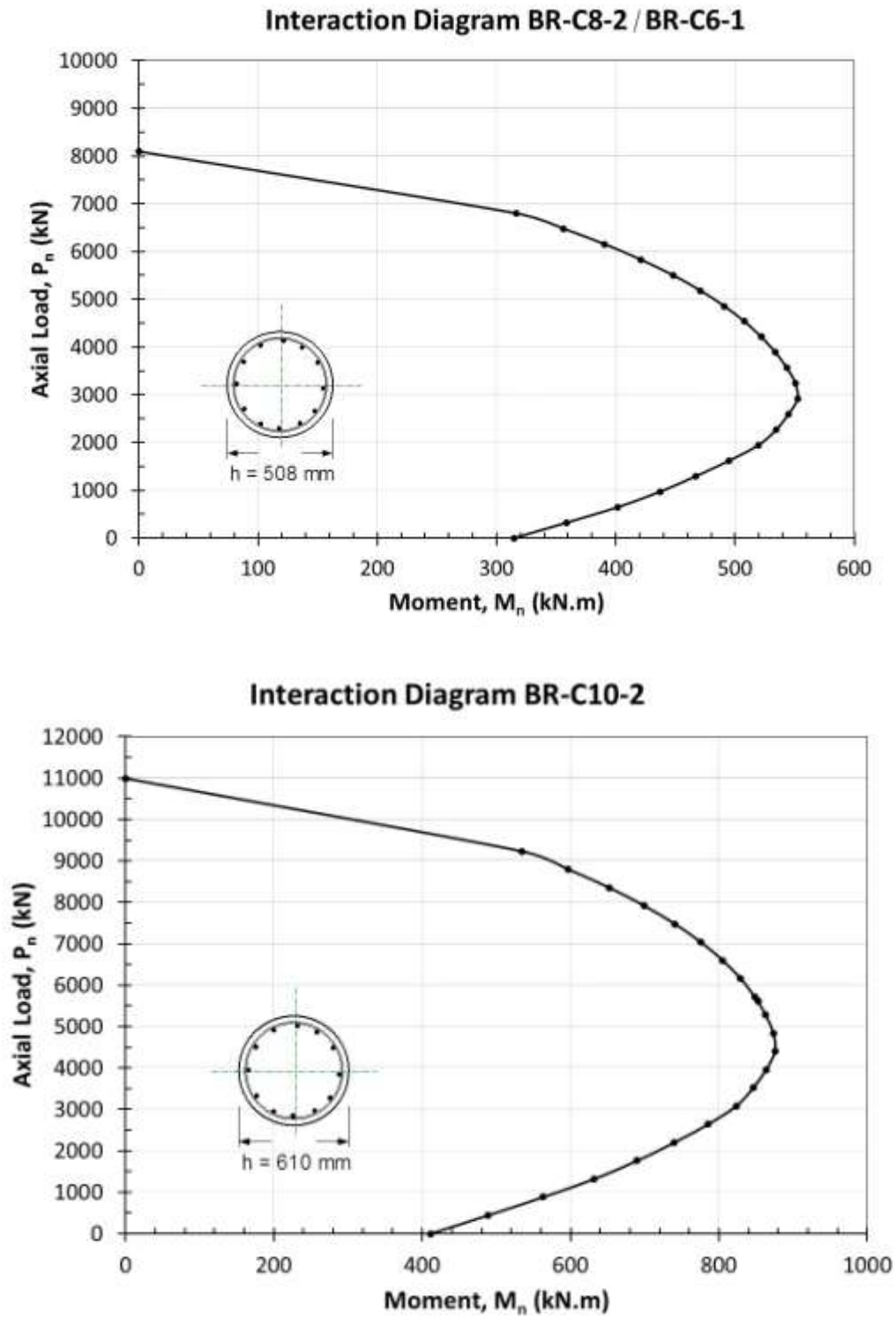
where,

$\Delta_1$  = Displacement measured by LVDT No. 1

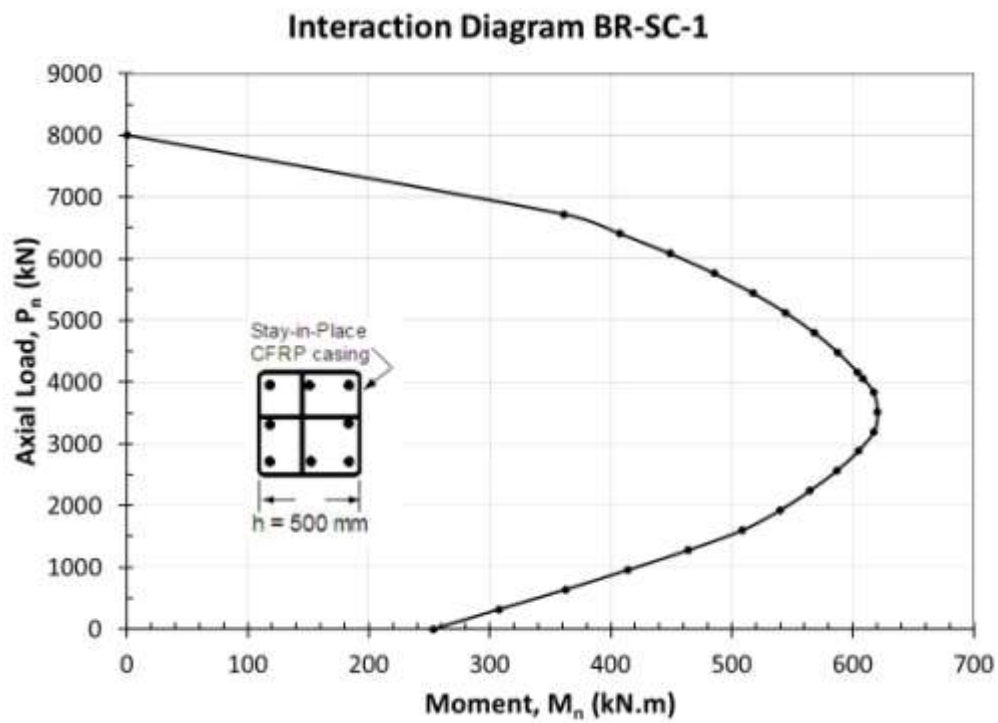
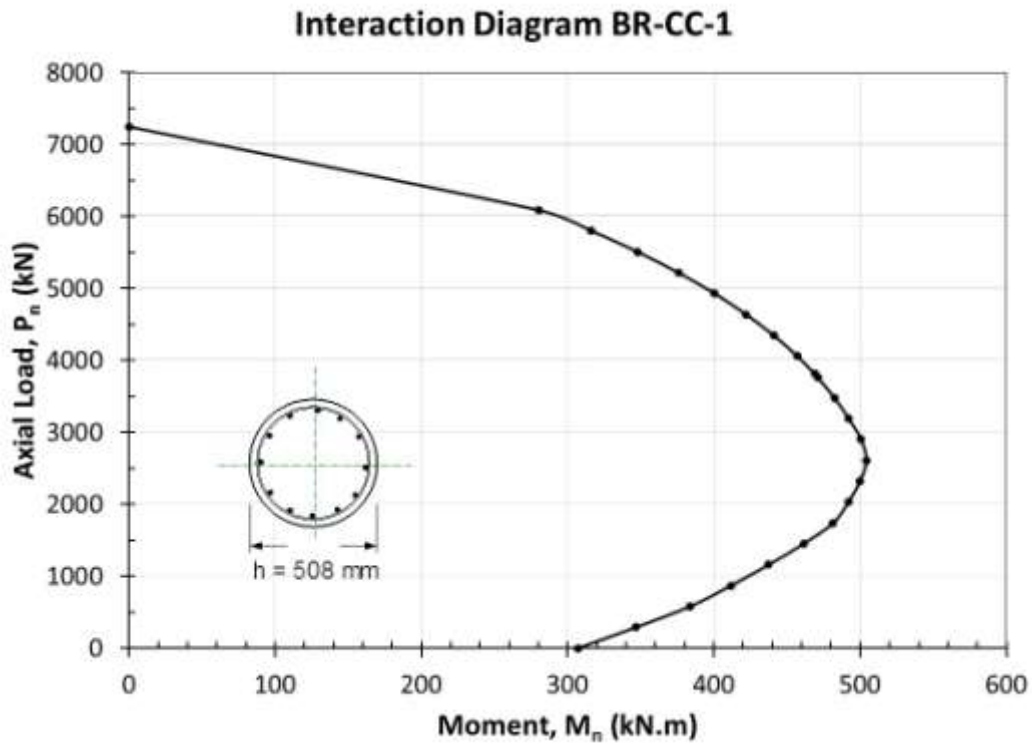
$\Delta_2$  = Displacement measured by LVDT No. 2

$d_{LVDT}$  = Horizontal distance between LVDT No.1 and LVDT No. 2

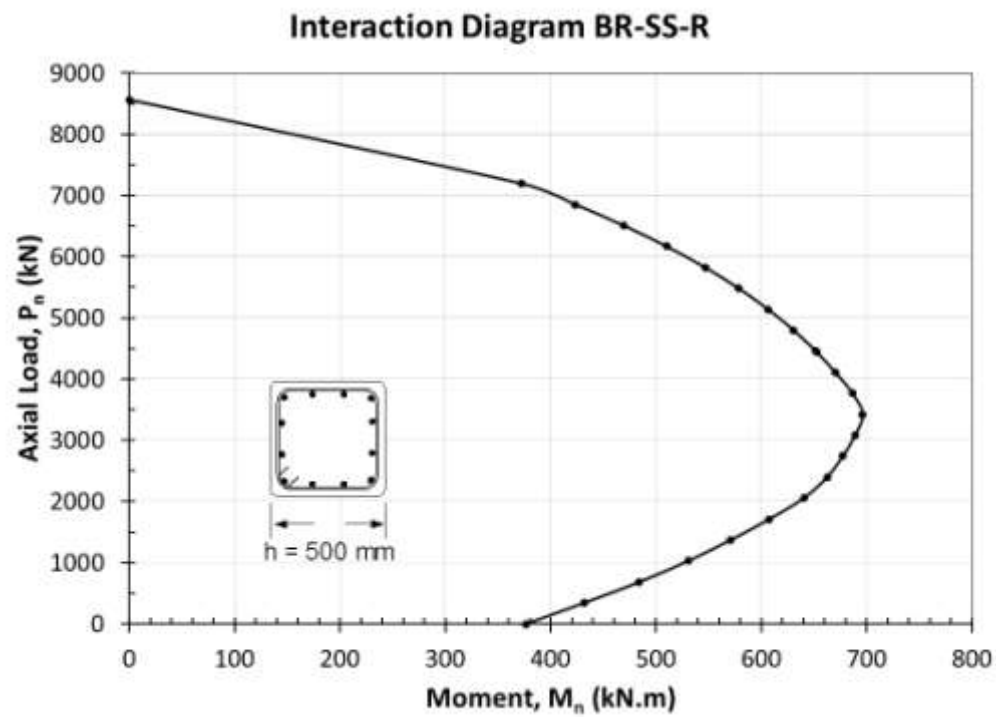
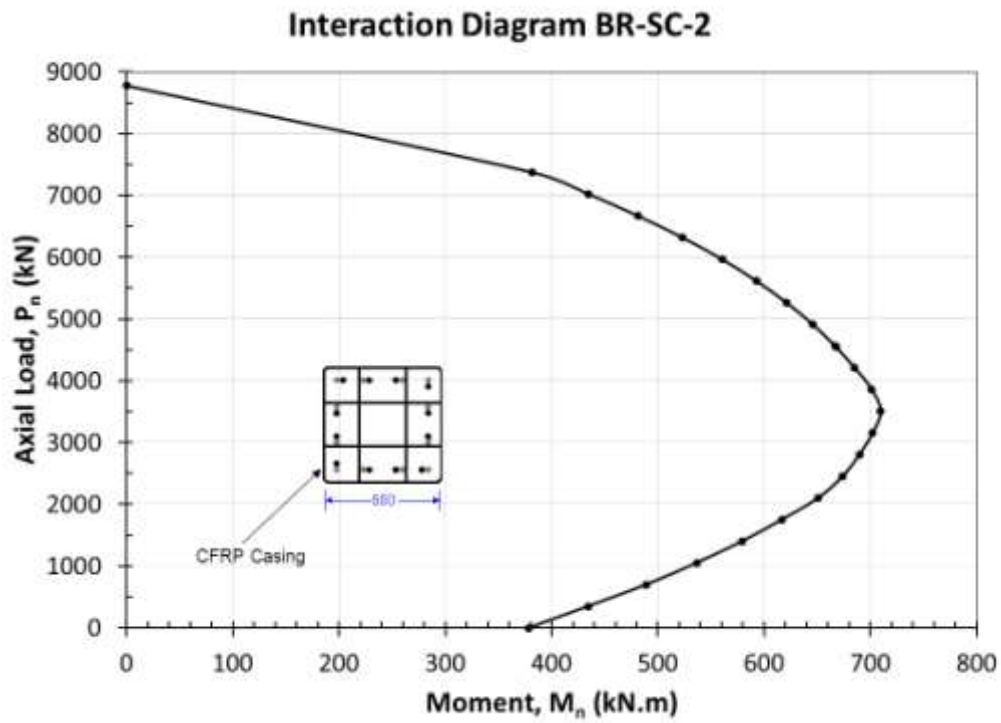
Two temposonic LVDT's were used to evaluate the anchorage slip rotation  $\theta_0$  at the column - footing interface. The same equation, as shown above, was used to compute rotations from recorded displacements. A third Temposonic LVDT was used to measure horizontal displacements of columns at the point of application of horizontal load corresponding to the effective height of column. Figure 2.32 shows the locations of LVDT's.



**Fig. 2.1** Interaction diagrams for test specimens



**Fig. 2.1 (contd.)** Interaction diagrams for test specimens



**Fig. 2.1 (contd.)** Interaction diagrams for test specimens



(a)



(b)

**Fig. 2.2** Construction of steel cage and formwork for column footings



(a)



(b)



(c)

**Fig. 2.2 (contd.)** Construction of steel cage and formwork for column footings





(a)



(b)



(c)

**Fig. 2.3** Column casting (BR-SS-R)





(a)



(b)



(c)



(d)

**Fig. 2.4** Column casting in stay-in-place casing (BR-CC-1& BR-SC-1)



**(a)** Composite system components



**(b)** Hardware used for installation

**Fig. 2.5** CFRP composite system and the necessary hardware



(a) BR-C8-2



(b) BR-C8-2



(c) BR-C10-2



(d) BR-C10-2

**Fig. 2.6** Columns BR-C8-2 and BR-C10-2 before and after primer application





(a)



(b)



(c)



(d)



(e)



(f)

**Fig. 2.7 CFRP sheet application**



(g)

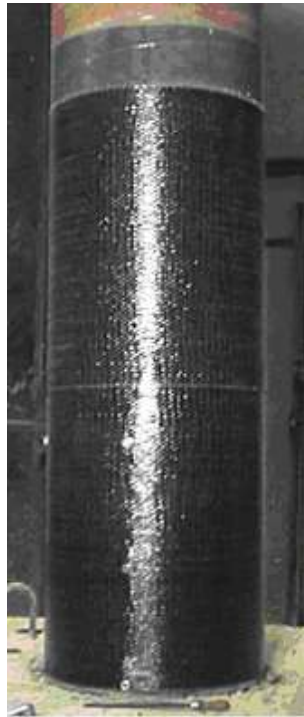


(h)

**Fig. 2.7 (Cont'd) CFRP sheet application**



(a) BR-C6-1



(b) BR-C8-2



(c) BR-SS-R

**Fig. 2.8 Completed retrofitted specimens**



(a)



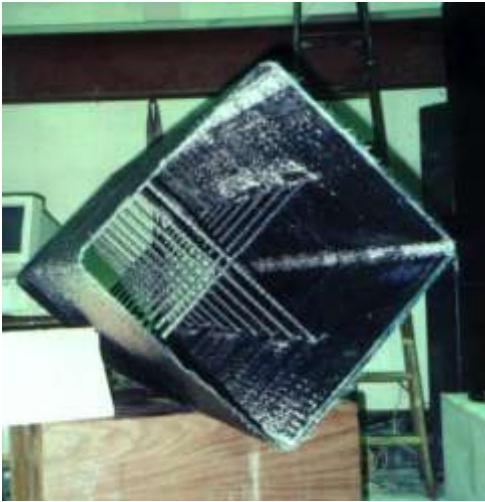
(b)



(c)

**Fig. 2.9** CFRP casing for column BR-CC-1





(a)



(b)

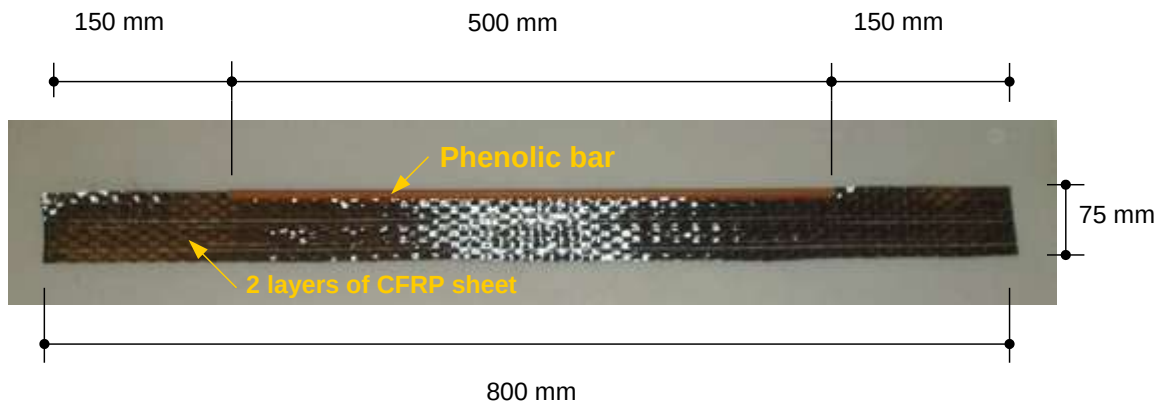


(c)



(d)

**Fig. 2.10** CFRP casing for column BR-SC-1



(a) Details of CFRP ties



(b) CFRP ties

**Fig. 2.11** Internal Crossties



(a)



(b)

**Fig. 2.12** CFRP ties integrated within the casing for column BR-SC-1





(a)



(b)



(c)



(d)

**Fig 2.13** Casing fabrication for square column BR-SC-2



(a)



(b)

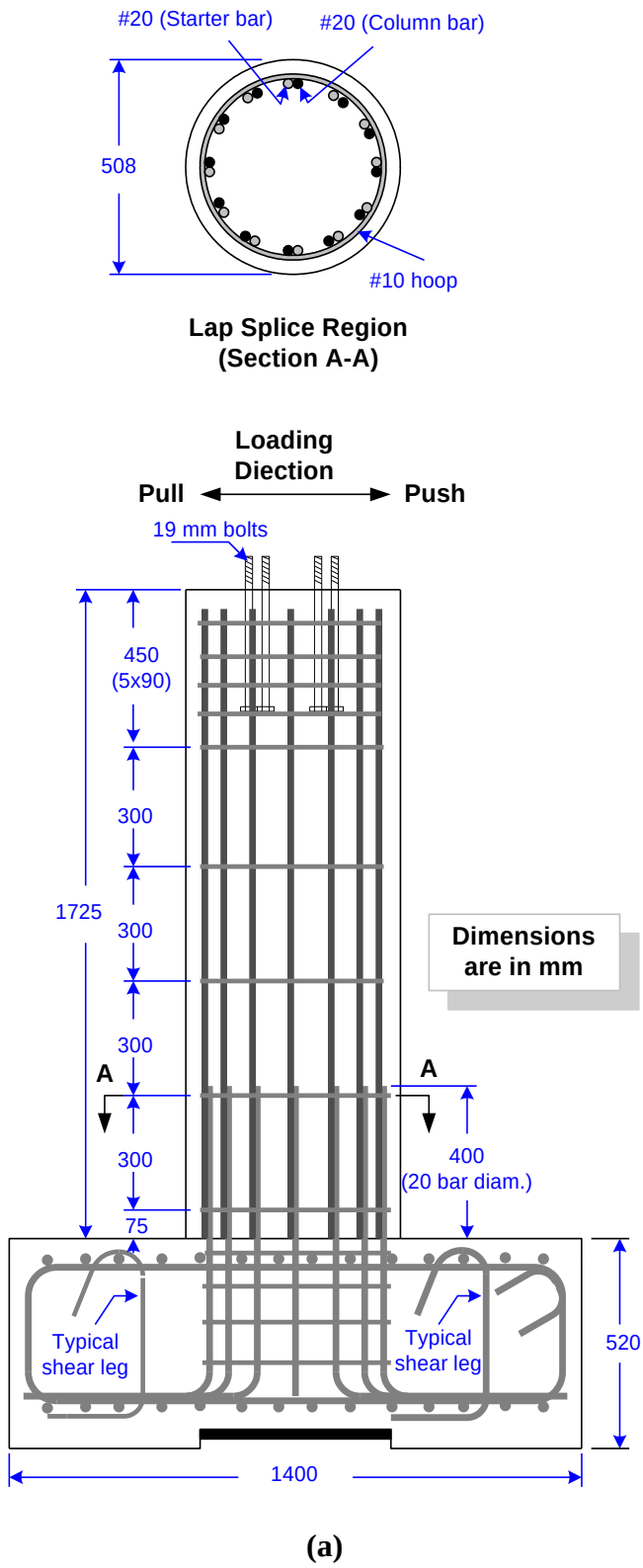


(c)



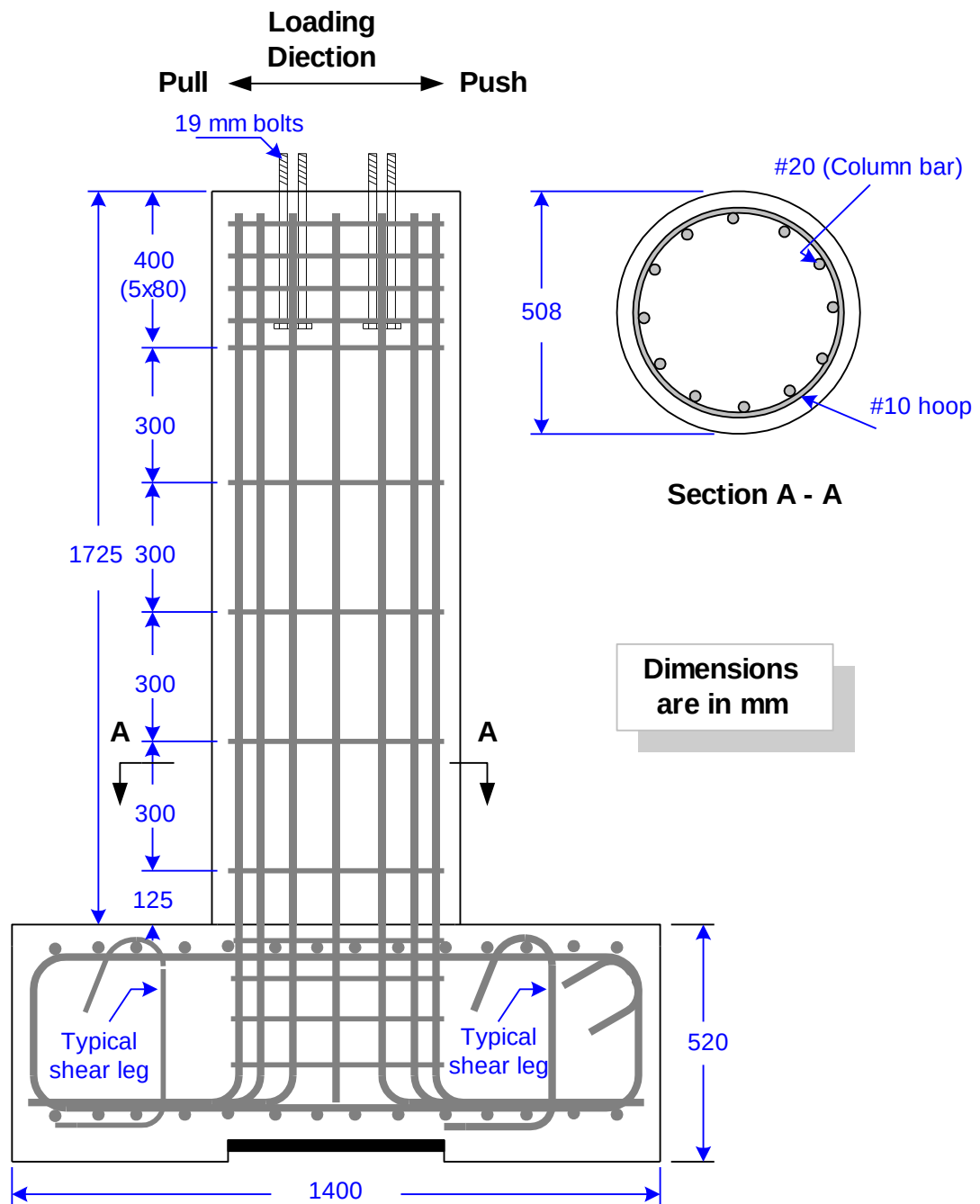
(d)

**Fig. 2.14** Construction of column BR-SC-2

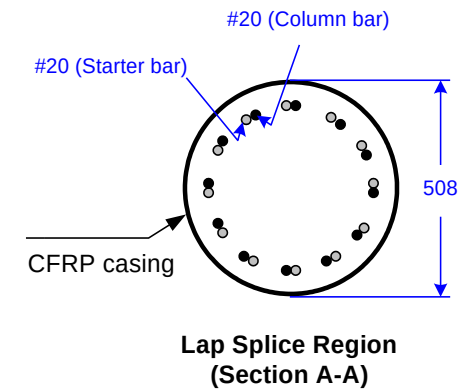


**Fig 2.15** Dimension and reinforcement details for column BR-C8-2

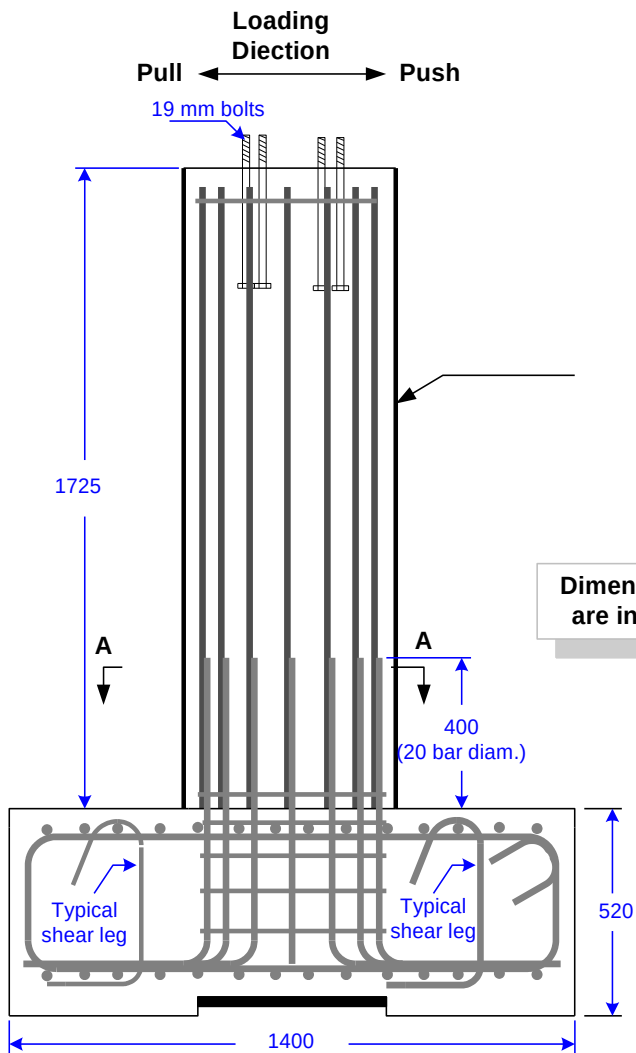




**Fig 2.17** Dimensions and reinforcement details for column BR-C6-1



**(b)**



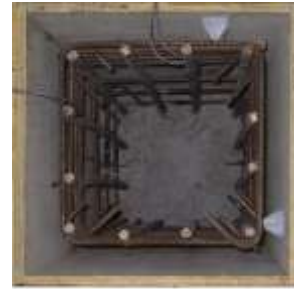
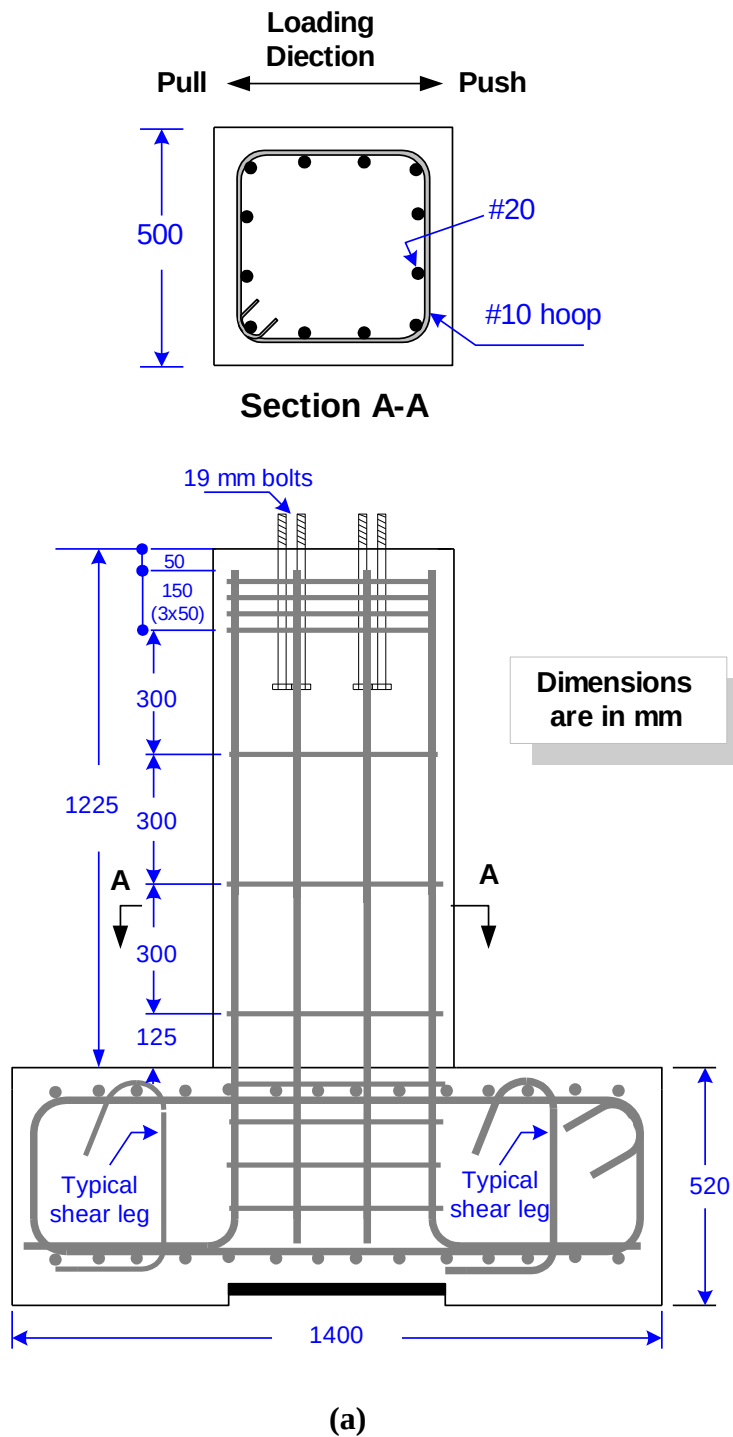
**(a)**



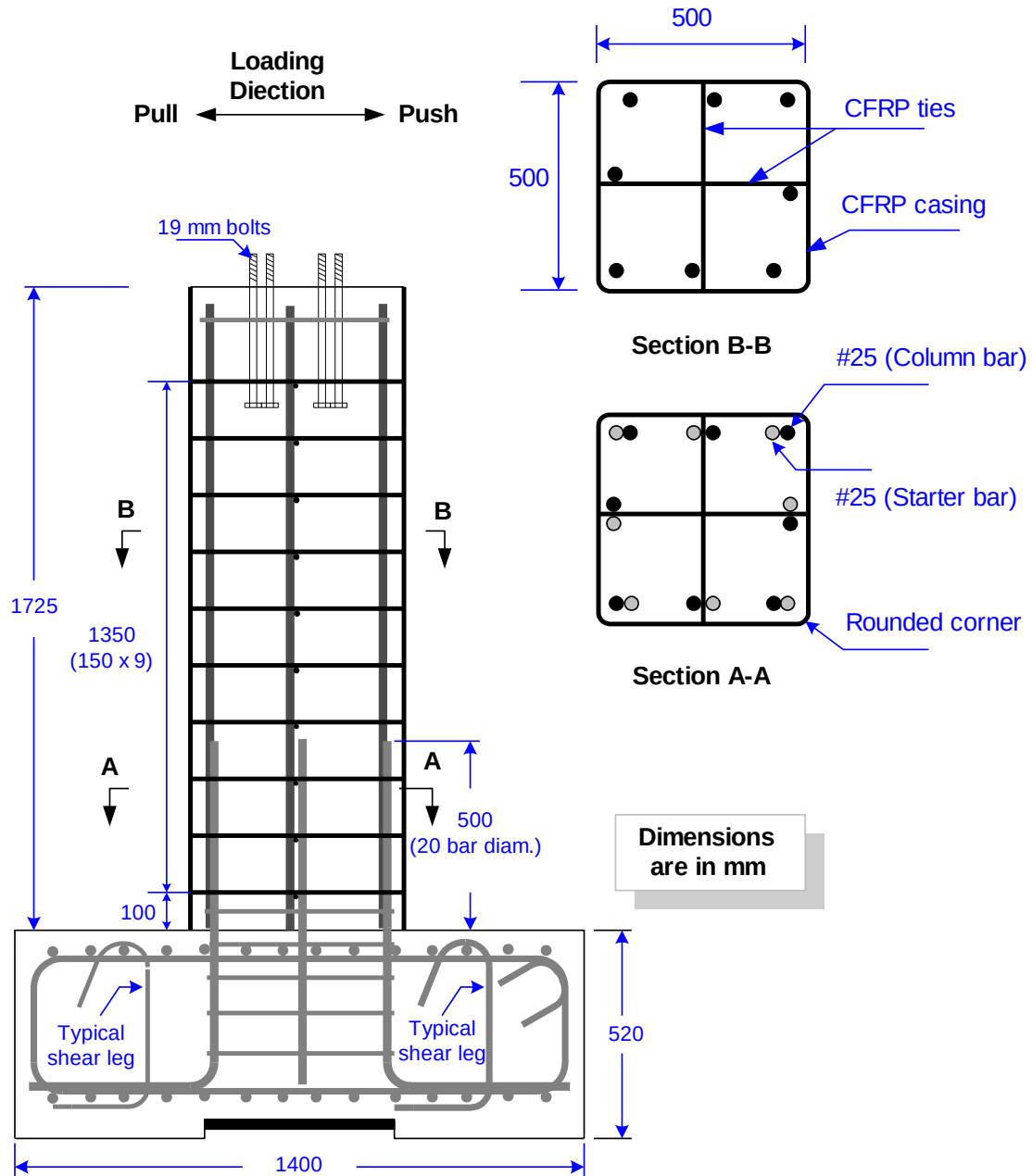
**(c)**

**Fig 2.18** Dimensions and reinforcement details of column BR-CC-1



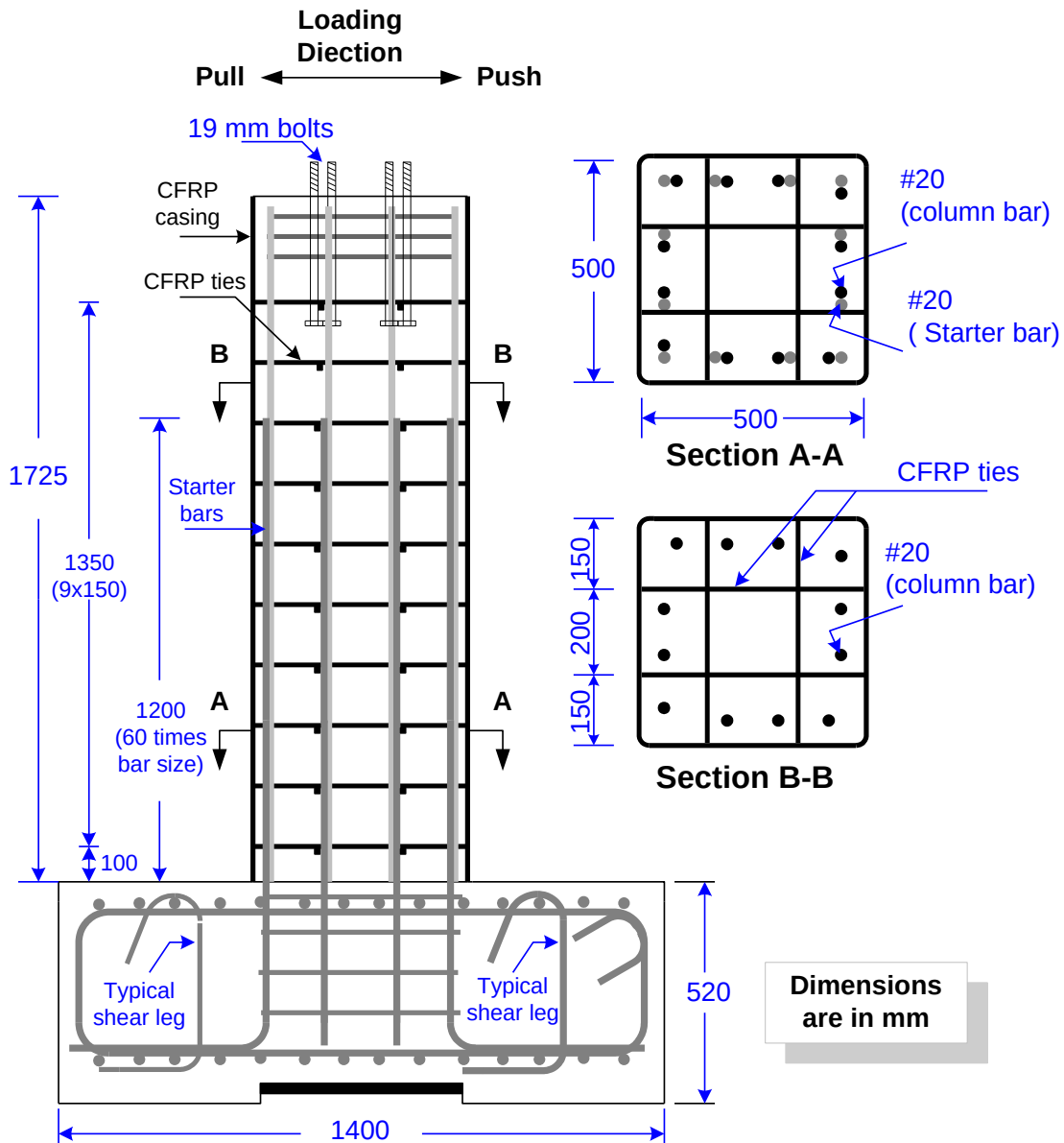


**Fig 2.19** Dimensions and reinforcement details of column BR-SS-R



**Fig 2.20** Dimensions and reinforcement details of column BR-SC-1





**Fig 2.21** Dimensions and reinforcement details for column BR-SC-2



**Fig. 2.22** FURNEY® Testing Machine



GALDABINI® Testing Machine

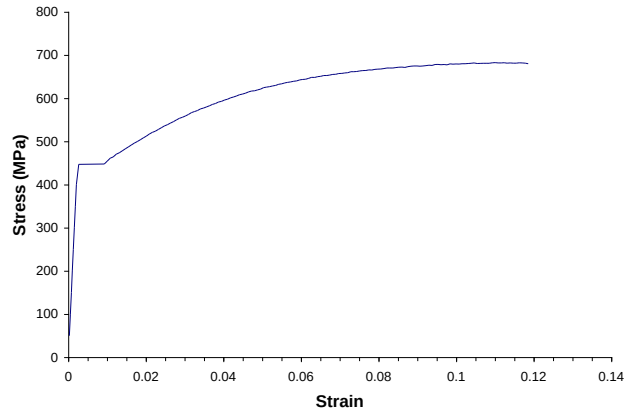


Sample ready for tension test

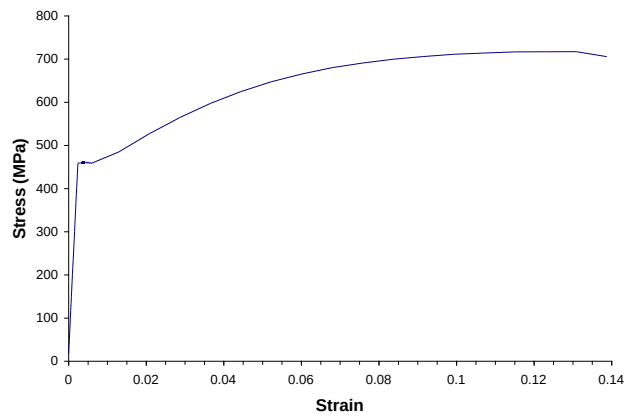


Deformed sample

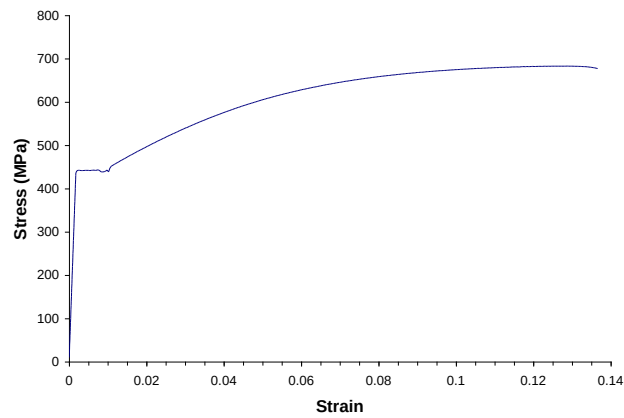
**Fig. 2.23** Reinforcing steel Coupon Test



Stress-strain relationship of grade 400-No.10 reinforcing steel bar

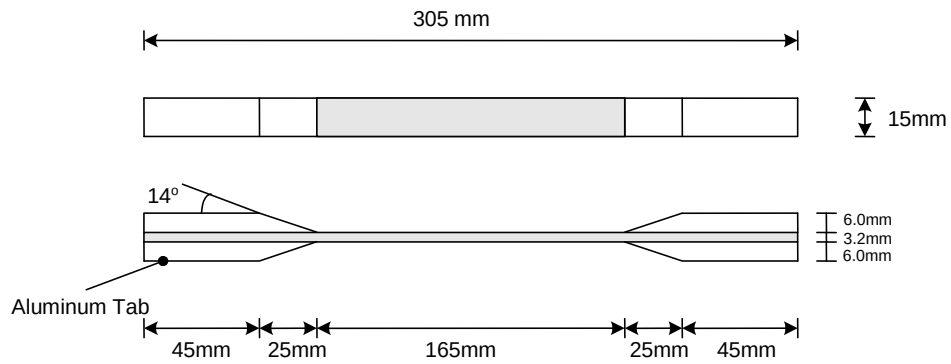


Stress-strain relationship of grade 400-No.20 reinforcing steel bar

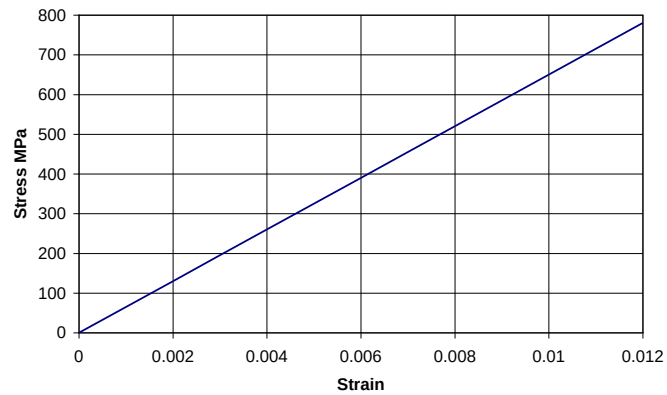


Stress-strain relationship of grade 400-No.25 reinforcing steel bar

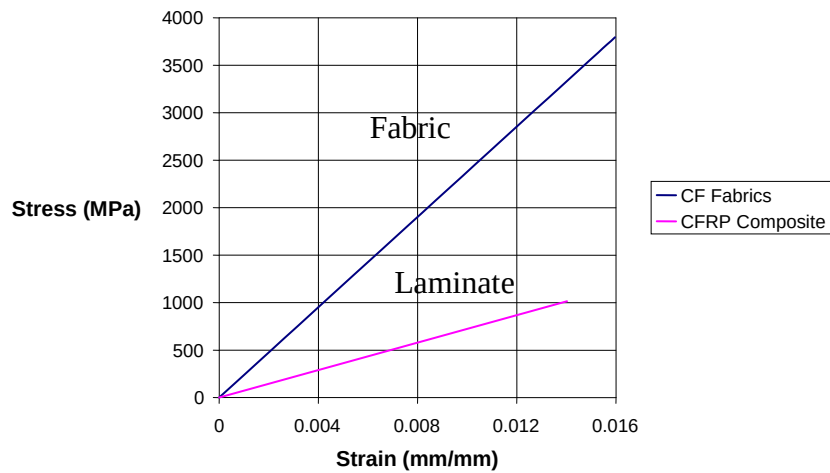
**Fig. 2.24** Stress-Strain relationship of reinforcing steel bars



**Fig. 2.25** Tensile Test Specimen (ASTM Standards)



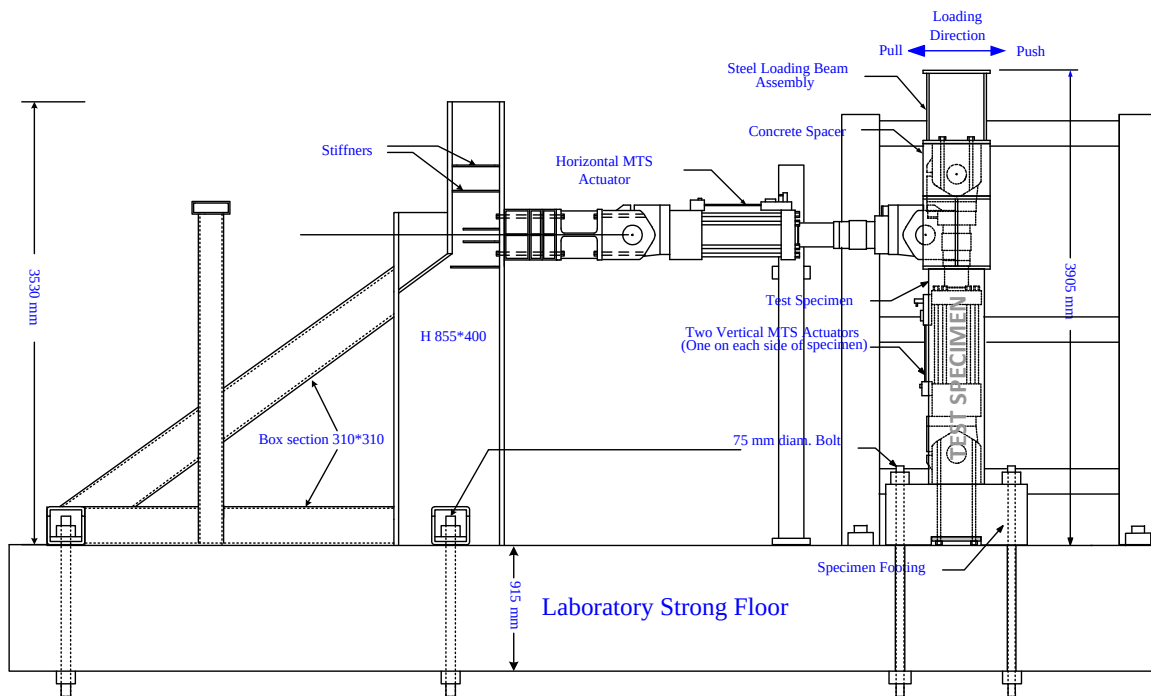
**Fig. 2.26** CFRP composite coupon test result



**Fig. 2.27** Ultimate tensile strength for Wabo MBrace CF 130 Carbon Fiber Fabric and CFRP laminate

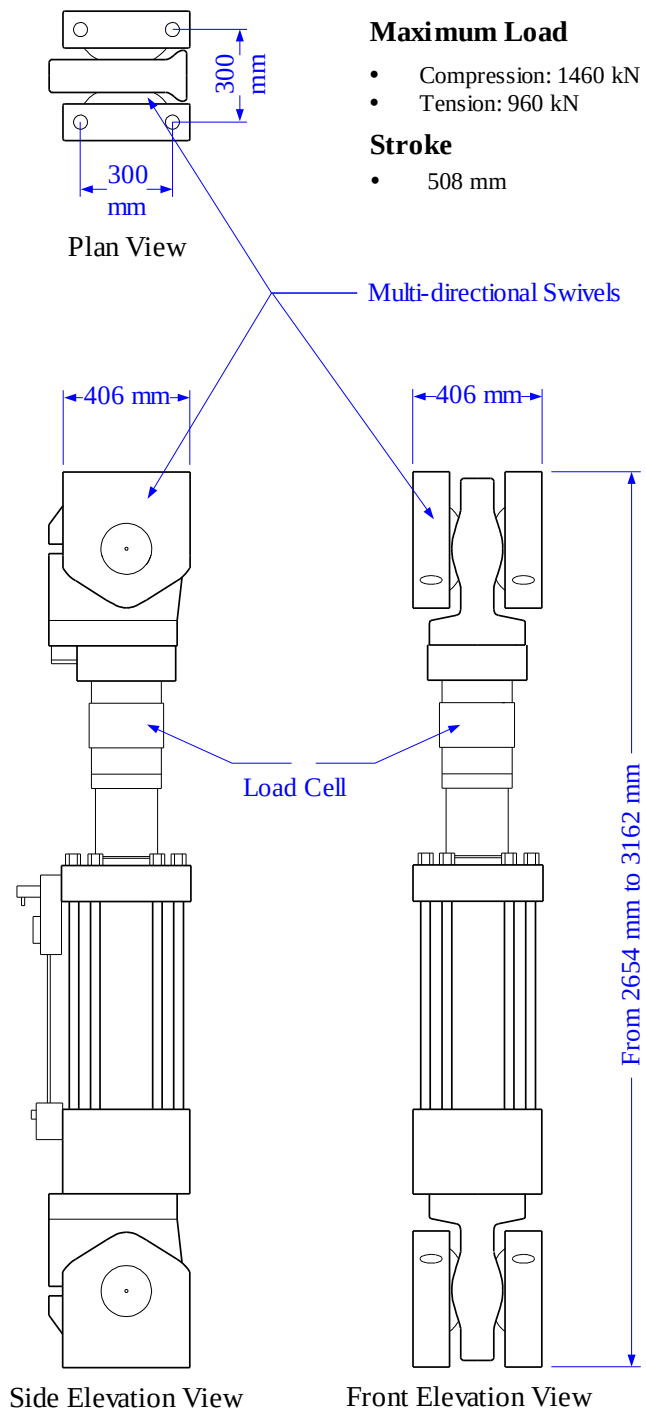


**Fig. 2.28** overall view of test set-up

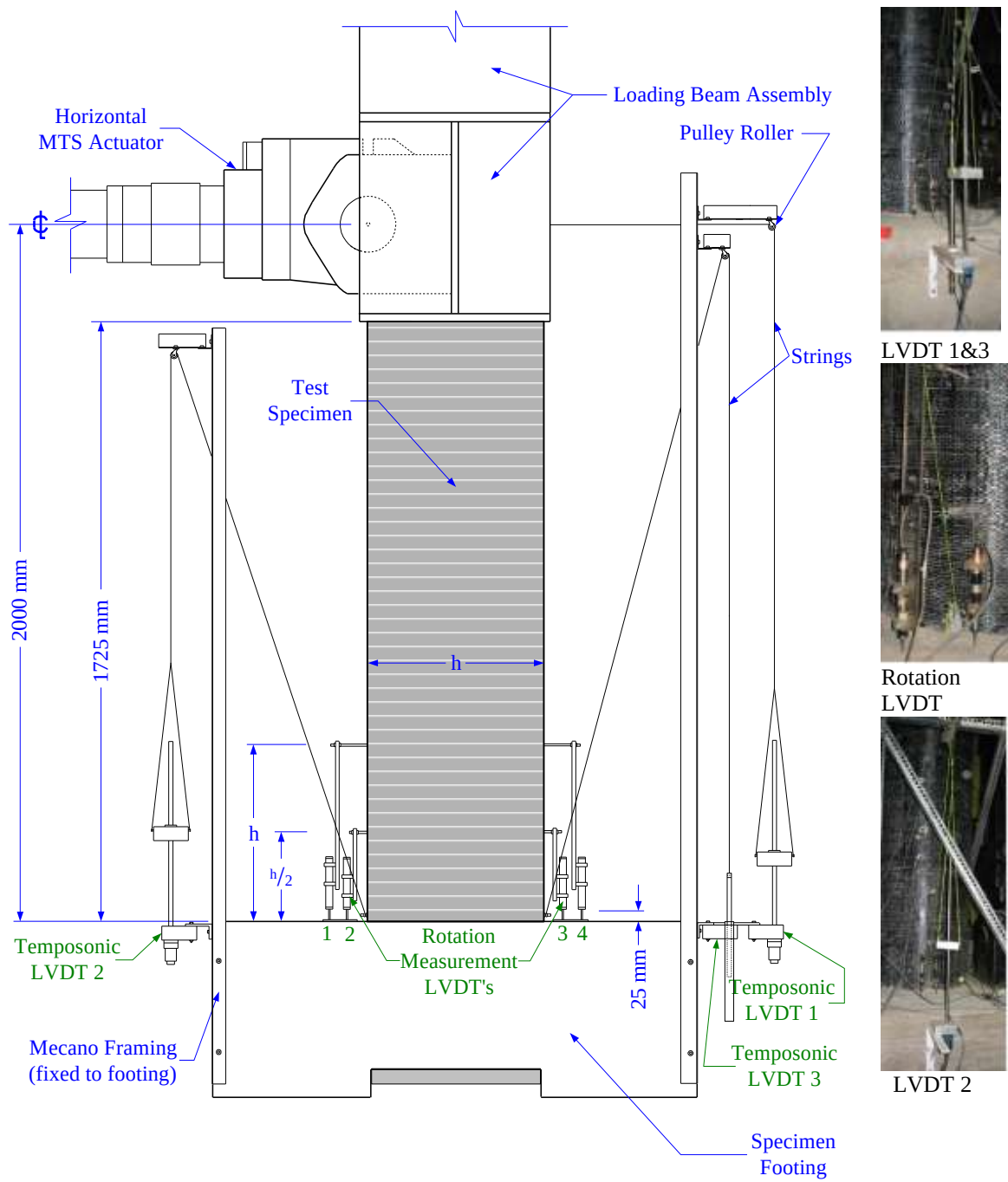


**Fig. 2.29** Side elevation of test set-up



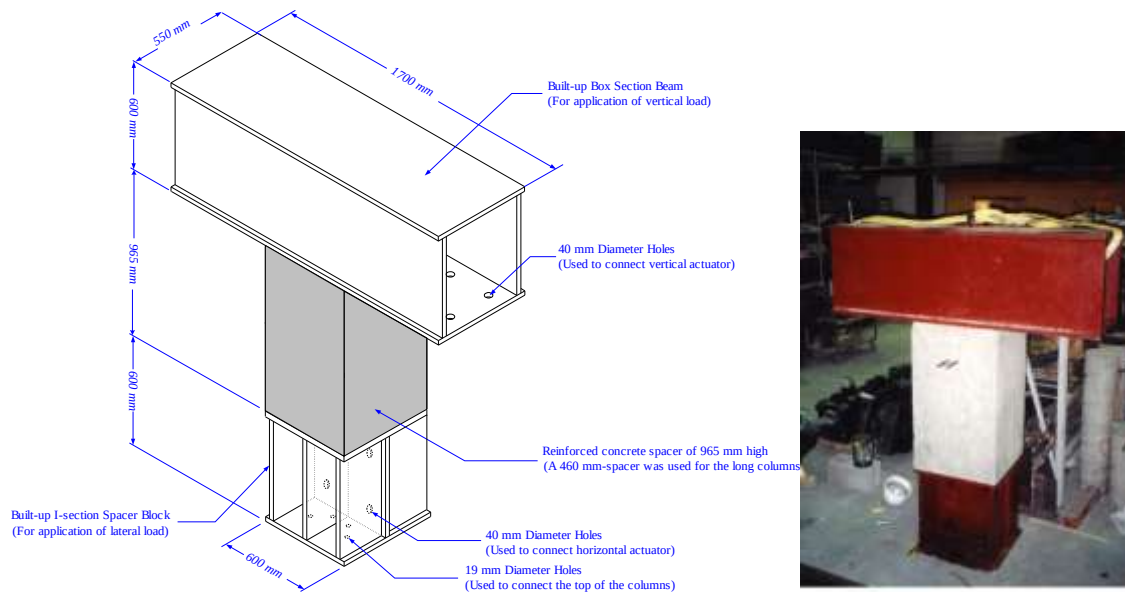


**Fig. 2.31** Details of the MTS hydraulic actuators

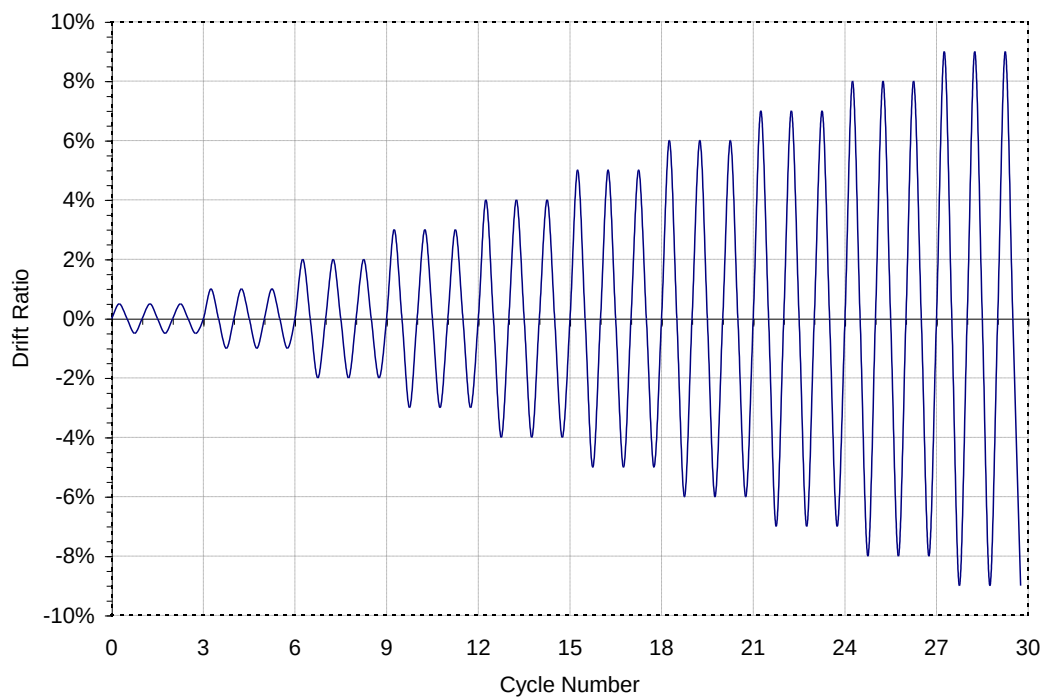


**Fig. 2.32** Location of LVDT





**Fig. 2.33** Details of the loading beam assembly



**Fig. 2.34** Loading Program

# CHAPTER 3

## EXPERIMENTAL RESULTS

### 3.1 GENERAL

The data obtained from column tests and the observations made during testing are summarized in this Chapter. A total of seven columns are presented to assess the effectiveness of CFRP sheets for jacketing existing bridge columns, as well as for manufacturing stay-in-place formwork for new columns.

The test data is compared with control specimens (unretrofitted), tested by other researchers at the University of Ottawa (Beausejour, 2000, Mes, 1999, Yalcin, 1998). The data is also compared with retrofitted columns tested by the author during an earlier phase (Phase I) of this research program (Elnabelsy, 2000). The columns are considered to have failed once the flexural resistance drops below 80% of peak resistance. Many researchers use this criterion as a means for deciding column capacity. The last drift level associated with such resistance (i.e. prior to strength decay below 80% of the peak) is referred to as the column *drift capacity*. The drift capacity is used to express inelastic deformation capacity of columns. The relative performance of columns is illustrated through Lateral Force - Drift and Moment - Drift hysteretic relationships. It should be noted that, the Moment-Drift relationships do not exhibit the strength decay in lateral force resistance associated with  $P-\Delta$  effects, though this effect can clearly be observed in the Force-Drift relationships.

The presentation of data includes strain gauge measurements. While great care has been taken in installing strain gauges, some of them were not able to record data up to the end of testing. The reliable strain measurements recorded on reinforcing steel and FRP elements (jackets/casings/ties) are included at the end of this Chapter. It should be noted that the positive sign in strain measurements indicates tensile strain and negative sign indicates compressive strain.

The test set-up was oriented such that the columns were loaded in the east-west direction. Cycles at each level of deformation started with a push towards west, followed by a pull towards east. Drift during pushing is shown as positive (right side of hysteretic relationships) and drift during pulling is shown negative (left side of the graphs).

Discussion of observed behaviour for each column is presented in the following sections.

### **3.1.1 LONG CIRCULAR COLUMN BR-C8-2**

Column BR-C8-2 had a 508 mm circular cross section and a 2.0 m shear-span, exhibiting predominantly flexural behaviour. The longitudinal reinforcement was spliced near the base to simulate the practice in existing bridge columns. The column was retrofitted with 2 plies of CFRP sheets. The behaviour of column at different drift levels is illustrated in Fig 3.1. The force-deformation hysteretic relationships are shown in Fig. 3.2, exhibiting ductile response.

The lateral load and moment resistances during the initial elastic deformation cycles of ½% drift ratio were 181 kN and 376kN.m, respectively. There was no distress observed at this early stage of loading. At 1% drift, the force and moment resistances increased to 224 kN and 471 kN.m, respectively. The column began yielding at this level, and the dowel strains reached 3000  $\mu\epsilon$  at the column-footing interface. Further loading to 2% drift ratio resulted in an increase in load resistance of up to 242 kN and a moment resistance of 520 kN.m. The strain readings indicated that the dowels on the East side entered into the strain hardening range, as strain gauge No. 9 exceeded  $\epsilon_{sh}$  of 1.2% or 12000 $\mu\epsilon$  during the first load excursion to 2% lateral drift. Signs of pinching of hysteresis loops started to appear at this stage of loading, signifying the onset of slippage of spliced reinforcement near the base. Slight strength decay was experienced during 3% drift cycles, and the load capacity decreased to 224 kN with a corresponding decrease in flexural capacity to 505 kN.m. Strain gauges 5 and 12 on the longitudinal reinforcement went off-scale, probably due to a wire cut associated with cracked and damaged concrete.

However, Strain Gauges 7 and 14 located at a height of  $h$  from the column-footing interface recorded near yield strains. The hoops yielded at this stage, with strain readings reaching 2000 to 3000  $\mu\epsilon$ . Pinching of hysteresis loops have become more pronounced at 3% drift ratio, indicating increased bar slippage at lap splices.

The column showed about the same resistance during 4% drift cycles, indicating the effectiveness of CFRP jacket in improving bond in the splice region and confining concrete. This was also evident in strain readings taken on the CFRP jacket. The strain reading at this drift level, at a distance 50 mm above the column-footing interface, showed 6000 $\mu\epsilon$  in the circumferential direction, and kept increasing linearly as testing progressed. The specimen showed signs of the separation of fibres along the circumference of column as depicted in Fig 3.1. This was attributed to the rupturing of threads or yarns that held the fibres together upon cracking of concrete underneath. The column experienced some strength decay when deformed to 5% drift ratio, developing a moment resistance of 495  $kN.m$  during the first push cycle, which dropped to 440 and 420  $kN.m$  during the 2<sup>nd</sup> and 3<sup>rd</sup> cycles, respectively, corresponding to 85 % and 81% of peak resistance. Although the moment resistance was 422  $kN.m$  during the first cycle at 6% drift, which was slightly higher than 80% of peak resistance, the resistance dropped to 396  $kN.m$ , corresponding to 75% of peak flexural resistance during the 3<sup>rd</sup> cycle at 6% lateral drift. The column was considered to have failed at this deformation level and the test was discontinued.

The rotation of column near the based was measured with pairs of vertically positioned LVDT's. Total rotations within distances of  $h$  and  $h/2$  relative to the column-footing interface, as well as the anchorage slip rotations are shown in Fig 3.3. The stable hysteretic relationships recorded indicate the effectiveness of CFRP jacket. However, the pinching of hysteretic loops, as well as the significantly high anchorage slip rotations measured (about 50% to 70% of total) indicates that the slippage of longitudinal reinforcement could not be prevented. The rotations were evenly distributed within a distance of  $h = 508\text{mm}$  from the base. At 5% drift level, the rotations measured were;  $50 \times 10^{-3}$ ,  $60 \times 10^{-3}$  and  $70 \times 10^{-3}$   $rad$  at  $h = 0, 254$ , and  $508$  mm, respectively.

The reinforcement for the column was instrumented with electric resistance strain gauges. Figs. 3.4 through 3.8 illustrate recorded strains on dowels, longitudinal bars, hoops and the FRP jacket. The hoop strains shown in Fig 3.6 indicate that the column transverse reinforcement remained elastic up to 2% lateral drift. Strain gauge No. 16 exceeds 2000  $\mu\epsilon$  at 3% drift and reached a maximum strain of 4500  $\mu\epsilon$  at 5% drift. Strain gauge No. 15 failed after 3% drift, but showed similar behaviour to Gauge No. 16. The hoops close to the top experienced lower strains and remained elastic throughout testing.

Fig. 3.7 shows recorded strains at different locations on the CFRP jacket. A maximum strain of 11000  $\mu\epsilon$  occurred while loading to 6% drift, at a location 50mm above the footing level. This value is close to the CFRP rupturing strain of 12000  $\mu\epsilon$  measured in coupon tests. Strain gauges located at higher levels showed lower strain values, varying between 500 to 2500  $\mu\epsilon$ . The strain distribution on CFRP sheets along the height of the column is illustrated on Fig 3.8.

The column failed due to the slippage of longitudinal reinforcement at high levels of lateral drift. Although the column sustained 5% drift, it was very close to 80% of its peak resistance during the 3<sup>rd</sup> cycle at this drift level. Accordingly, the drift capacity of the specimen was considered to be 4%.

### **3.1.2 SHORT CIRCULAR COLUMN BR-C10-2**

Column BR-C10-2 had a 610 mm circular cross-section and a shear-span of 1.5 m. The column main reinforcement was spliced in the hinging region and the retrofitting strategy consisted of 2 layers of CFRP sheets wrapped along the circumference of the specimen. Fig 3.9 illustrates the behaviour of column at different drift levels. Fore-Drift and Moment-Drift hysteretic relationships are shown in Fig 3.10.

Testing started by pushing the column to ½ % drift, reaching a maximum moment of 570 kN.m, which correspond to 449 kN of lateral loading. Dowel strains recorded at this

deformation level were close to  $2000\mu\epsilon$  indicating a possible yielding during the next drift level.

The column was pushed to 1% drift, developing a flexural capacity of 680 kN.m. The dowels yielded at this stage of loading, reaching a strain level of  $3000\mu\epsilon$  as recorded by Strain Gauges 2 & 9 installed at the column-footing interface, and  $2500\mu\epsilon$  measured by Strain Gauges 1 & 8 located inside the footing. Longitudinal reinforcement yielded at this drift level as well, with Strain Gauge 7 at a location 320 mm above the footing reaching  $2500\mu\epsilon$ .

The resistance continued to increase during the first cycle at 2% drift, reaching a peak moment of 705kN.m and corresponding load of 441kN. The column started to show some loss in strength during the second and third cycles of 2% drift, with recorded moment resistances of 657 kN.m and 625 kN.m, respectively. The strain readings on all longitudinal reinforcement remained constant at the end of the first cycle at 2% drift, indicating the slippage of longitudinal bars relative to the starter bars.

At 3% drift, the resistance was initially 664 kN.m and dropped to 629 kN.m and 600 kN.m in the second and third cycles. The loss in resistance can be observed in strain trends recorded by Gauges 7 & 14, which indicate very little bond generated to prevent the slippage of lap splices. This is also verified by the hoop strains which remained approximately constant beyond 3% drift until the end of testing. The test was continued, and the column was pushed to 4% drift with 630 kN.m. flexural resistance and continued showing strength decay with 600 kN.m and 563 kN.m moment resistances in the subsequent two cycles. The latter resistance corresponded to 80% of peak resistance, indicating potential failure during the following cycle. Excessive crushing and spalling of cover concrete, as well as the crushing of core concrete was observed.

During the first cycle at 5% drift, the resistance was 540 kN.m, which was less than 80% of peak resistance. The substantial strength decay observed was attributed to the buckling of compression bars and subsequent loss in bond strength within the splice region. This is

clear in the pinched loops observed in the hysteretic relationship. The pinching became more pronounced when the column was pushed to 6% drift. The strength decay was very significant at this deformation level and testing was discontinued.

The rotations of the plastic hinge region as well as those caused by anchorage slip are shown in Fig 3.11. LVDT 2 and LVDT 3 were used to evaluate the total rotation at  $h$  from the column-footing interface. LVDT 2 malfunctioned, which led to some unreliable data on the push side. Therefore, the data on the pull side was used for both sides, assuming similar behaviour for both LVDTs. Based on this assumption, much of the rotation took place between  $\theta_0$  and  $\theta_{h/2}$  (254 mm from the column base), as the graphs for  $\theta_h$  and  $\theta_{h/2}$  are close in magnitude.

The strain data recorded on dowels and longitudinal reinforcement, as well as the strain gauge locations are shown in Figs. 3.12 and 3.13. The strain data recorded on the first hoop showed unreliable readings beyond 2% drift (Fig 3.14). From the trend of the graphs for Strain Gauges 16 & 15, it has become clear that the hoops started yielding during the first cycle at 2% drift, which explains some of the adhesion problems with the strain gauges. However, the upper hoop (Strain Gauges 18 & 17) indicated well below yield values, and de-bonding of the gauges was observed beyond 3% drift.

The strain values recorded on CFRP sheets are shown in Fig 3.15. The first level of strain gauges, placed at a distance 50 mm from the column-footing interface, showed higher strains than those recorded at other locations. These strains increased gradually reaching a maximum value of  $7500 \mu\epsilon$  at 6% drift (Strain Gauge 23). Strain gauge 26, placed perpendicular to the direction of loading, recorded a strain of approximately  $3000 \mu\epsilon$  at 4% drift before going off-scale. The other gauges, located at higher locations above the footing, recorded strain levels in the range of  $1000 \mu\epsilon$  to  $2000 \mu\epsilon$ . Strain distribution on CFRP sheets along the column height is illustrated in Fig. 3.16.

The column failed during the first cycle at 5% drift due to the slippage of splices reinforcement. Bar buckling and concrete crushing followed. The drift capacity of the specimen was established to be 4%.

### **3.1.3 LONG CIRCULAR COLUMN BR-C6-1 (CONTINUOUS BARS)**

Column BR-C6-1 had a 508 mm circular cross-section and a shear span of 2.0 m. Unlike the previous two columns, the longitudinal reinforcement was continuous, without the lap splices. The column was retrofitted with 2 layers of CFRP sheet. Its behaviour at different drift levels is illustrated in Fig 3.17. Fig. 3.18 illustrates Force-Drift and Moment – Drift hysteretic relationships.

During the initial cycles at  $\frac{1}{2}$  % drift the column developed 300 kN.m of flexural resistance, which corresponded to 150 kN of lateral force. Strain Gauge 5 on vertical reinforcement at column-footing interface did not function because of damaged wiring. However, Strain Gauge 2 at the same level, but on the pull side, recorded strain of  $1900\mu\epsilon$ , indicating near yield strain levels. The bars at column-footing interface yielded at 1% drift, and entered into strain hardening, before Strain Gauge 2 stopped working. The penetration of yielding into the footing was observed with strains reaching  $2100\mu\epsilon$  while moment resistance increased to 460 kN.m.

The moment capacity continued increasing with deformations reaching 522 kN.m and 554 kN.m at 2% and 3% drift ratios, respectively. At 3% lateral drift, the first hoop (at a distance 125 mm above the footing) approached yielding with strain readings of about  $2100\mu\epsilon$ . The column exhibited excellent behaviour at higher drift levels, with further increase in moment resistance; up to 584kN.m at 4% and 600 kN.m at 5% drift ratios. The latter was the peak load resistance attained during testing. The improved deformability was attributed to the effectiveness of carbon fibres in confining concrete, which maintained their integrity and developed significant hoop tension. This was evident by higher strains recorded for CFRP sheets, reaching  $6000\mu\epsilon$  at 5% lateral drift ratio.



Beyond the peak moment capacity, the column started losing some of its resistance, developing 580 kN.m, 564 kN.m, and 538 kN.m at 6%, 7%, and 8% drift ratios, respectively. Although the strength never dropped below 80% of peak, the test was stopped due to the stroke limit of actuator.

The hysteretic relationship for hinging region rotation is shown in Fig 3.19. Two Temposonic LVDT's were used to measure anchorage slip rotations ( $\theta_o$ ) and two pairs of LVDT's were used to measure total rotations at  $h/2$  and  $h$  ( $\theta_{h/2}$  and  $\theta_h$ ), as explained in the previous Chapter. One of the LVDT's measuring total rotations at distance  $h$  (LVDT 2) developed technical problems beyond 2% drift, because of stroke limitations. Therefore, the only rotation data available for this segment was from LVDT 4 on the push side. Assuming symmetric behaviour, i.e., the two opposite LVDT recordings being the same, the rotation data was extrapolated beyond the 2% drift level. This assumption was confirmed to be valid, as the readings obtained by LVDT 2 up to 2% drift were close to those recorded by LVDT 4. The rotation data indicated that at 5% drift level,  $\theta_o$ ,  $\theta_{1/2h}$  and  $\theta_h$  were 0.017 rad, 0.034 rad, and 0.51 rad respectively.

The strains recorded on steel bars and CFRP sheets are plotted in Figs 3.20 through 3.22. Figure 3.22 indicates CFRP strains of up to  $2500\mu\epsilon$  at 3% drift ratio at 50 mm above the footing (Strain Gauge 13). The strain values were below  $1200\mu\epsilon$  at 250 mm above the footing (as recorded by Strain Gauges 14 & 18). As indicated above, the CFRP strains developed  $6000\mu\epsilon$  at 5% lateral drift, at a height of 50 mm above the footing. This strain value kept increasing as lateral drift ratios increased and reached  $10000\mu\epsilon$  at 8% drift ratio. The CFRP strains recorded further higher up along the column were below  $500\mu\epsilon$  to  $3500\mu\epsilon$ , at the same stage of loading. Strain distribution on CFRP sheets along the column height is illustrated in Fig. 3.23.

The column sustained high drift ratios, exhibiting ductile behaviour, without significant strength decay. The energy dissipation capacity was greatly enhanced. The drift capacity was judged to be 8%.

### 3.1.4 SHORT SQUARE COLUMN BR-SS-R (CONTINUOUS BARS)

BR-SS-R is a 500 mm square, shear dominant column having a shear span of 1.5 m with continuous longitudinal reinforcement. Fig 3.24 illustrates the behaviour of column at different stages of testing. Lateral force – drift and moment – drift hysteretic relationships recorded during testing are shown in Fig. 3.25.

The test started by pushing the column to  $\frac{1}{2}$  % drift. The maximum recorded moment at this stage of loading was 460kN.m, corresponding to 306 kN of lateral force. The longitudinal reinforcement reached  $1900\mu\epsilon$  at the column-footing interface. The strain level increased to  $2700\mu\epsilon$  when the column was pushed to 1% lateral drift. The flexural capacity at this stage of loading was 530kN.m. The longitudinal reinforcement developed  $2800\mu\epsilon$  strain at a section 125 mm above the footing (Strain Gauge 3), indicating yielding.

During the next deformation level of 2% drift, yield penetration occurred into the footing and the embedded part of the longitudinal reinforcement inside the footing reached  $2800\mu\epsilon$  while the section just above the footing developed strain hardening in steel. The flexural capacity was 600kN.m, corresponding to a lateral force of 381 kN. The FRP sheets developed a strain level of  $1500\mu\epsilon$  at the bottom of specimen, and  $500\mu\epsilon$  at a location 250mm above the footing. The strain gauges placed on FRP, perpendicular to the direction of loading, showed small transverse strains. No significant strains were recorded on transverse reinforcement. This may be explained by the effect of CFRP sheets in resisting lateral pressure and maintaining the integrity of column concrete.

During the next deformation level of 3% lateral drift the resisting moment increased to 610kN.m and the longitudinal reinforcement inside the footing developed strain hardening. The column continued to show good behaviour with increased drift ratios, reaching a moment capacity of 612kN.m at 4% drift ratio and a peak flexural capacity of 613kN.m at 5% drift ratio. Signs of tearing in FRP layers were noticed, as shown in Fig 3.24. The fibre strain was  $2200\mu\epsilon$  at 5% drift ratio as recorded by strain gauge No. 15

while strain gauge No.19 & 20 installed on the East side of the specimen recorded  $2500\mu\epsilon$  and  $3000\mu\epsilon$ . The bulging of the casing observed on column sides (Fig 3.24) was believed to account for such increase in fibre strains.

At 6% drift, the column started losing some of its strength, although not substantially, with a drop of flexural capacity from 600kN.m to 534kN.m during the 3<sup>rd</sup> cycle. Bottom fibre strains reached  $3500\mu\epsilon$  on both sides of the column while the upper hoop (strain gauge No. 10) had a maximum strain of  $1400\mu\epsilon$ . The column sustained 521kN.m during the first cycle of 7% drift ratio before it failed during the second cycle due to the rupturing of longitudinal reinforcement. The three broken lines appeared in the force-drift relationship indicate the three ruptured bars.

Hysteretic relationships for anchorage slip rotations are illustrated in Fig 3.26. The hysteresis loops show well-rounded curves, indicating well-confined column behaviour. The rotation measurements were taken at two locations along a distance equal to column dimension “h” from the column-footing interface. At 5% drift, the anchorage slip was 0.002 rad with 0.015 increments for higher locations, resulting in 0.035 rad and 0.050 rad for  $\theta_{h/2}$  and  $\theta_h$  respectively. At 6% lateral drift, the anchorage slip was 0.0025 rad, increased to 0.040 rad for  $\theta_{h/2}$  and 0.055 rad for  $\theta_h$ .

Recorded strains for both longitudinal and transverse reinforcing steel are shown in Figs. 3.27 and 3.29. The progression of yield penetration into the footing is shown in Fig 3.28. Locations of strain gauges on FRP, as well as recorded strains, are shown in Fig. 3.30. Strain distribution on CFRP sheets along the column height is illustrated in Fig. 3.31.

In conclusion, the specimen behaved very well under cycling loading, and sustained large drift levels while maintaining its strength. The column failed during the second cycle of 7% drift ratio due to the rupture of longitudinal reinforcement. The drift capacity of the specimen was considered to be 6%.

## 3.2 COLUMNS WITH PREFABRICATED CFRP CASINGS

### 3.2.1 LONG CIRCULAR COLUMN BR-CC-1

Specimen BR-CC-1 was prepared using a prefabricated CFRP stay-in-place formwork. It was comparable to Specimen BR-C8-2 which was retrofitted with a CFRP jacket, in terms of column dimensions and cross sectional and longitudinal reinforcement properties. Both specimens had sufficiently high shear spans to behave predominantly in the flexure mode with 500 mm circular cross-sections. Both columns had spliced longitudinal reinforcement in potential plastic hinge regions. A major difference was in transverse reinforcement. BR-CC-1 was cast using a prefabricated CFRP mould consisting of 3 layers of carbon fibres. This stay-in-place mould was intended to act as continuous transverse reinforcement. No hoops were provided along the height of the column with fibres resisting the entire lateral force.

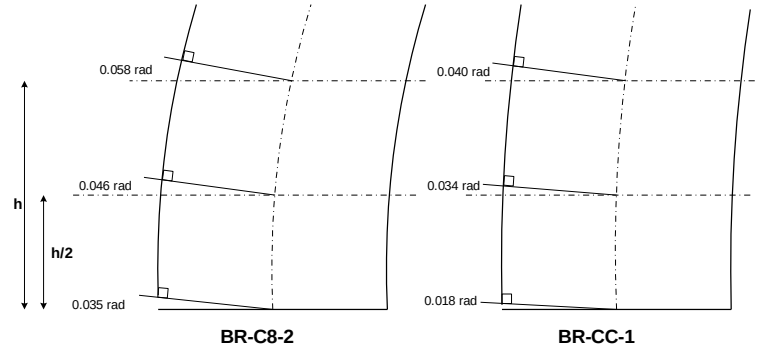
Fig 3.32 illustrates the column behaviour at selected stages of testing. The test started with pushing the column to  $\frac{1}{2}$  % drift ratio, reaching a flexural capacity of 351kN.m. The dowel bars developed a maximum strain of  $1800\mu\epsilon$  at this stage of loading, recorded at the column-footing interface. At 1% drift ratio, the resistance increased to 461kN.m. At this level, the dowels yielded and entered into strain hardening before strain gauges (No. 2 & 9) went off-scale. It is possible that the dowels exceeded  $\epsilon_{sh}$  of 1.2% or  $12000\mu\epsilon$  at this level. The test was continued and the column was pushed to 2% drift ratio, at which load stage the lateral load resistance reached 247kN and the moment resistance reached 530kN.m. Signs of pinching started appearing in the hysteretic relationship. Figure 3.33 shows Lateral Force-Drift and Moment-Drift hysteretic relationships. At 3% drift, the load capacity dropped slightly to 241 kN with a flexural capacity of 535kN.m, which indicated some strength decay. The longitudinal reinforcement showed low strains near the column-footing interface, reaching  $1250\mu\epsilon$  as recorded by strain gauge No. 5&12. The recorded strain was higher for strain gauges located at a distance 508 from the column-footing interface (strain gauges No. 7&14), reaching a value of  $1830\mu\epsilon$ . This was attributed to the crushing of concrete observed near the bottom of the column,

compared to the intact concrete at 508 mm above. Pinching of hysteresis loops observed in the force-deformation relationship confirmed the slippage of reinforcement, which became obvious beyond 3% drift ratio. At 4% drift, the column developed further strength decay with a recorded flexural capacity of 511kN.m during the first cycle, which dropped to 454 kN.m during the 3<sup>rd</sup> cycle. At this drift level, the strain in CFRP casing at a distance 50 mm measured from the footing-column interface reached  $3500\mu\epsilon$ . At distance  $h/2$  (250mm), the strain was  $2500\mu\epsilon$ .

The column was pushed to 5% drift ratio, reaching a moment resistance of 466 kN.m during the first cycle, and 411 kN.m during the 2<sup>nd</sup> cycle, corresponding to 77 % of peak resistance. The column was considered to have failed at this point and the test continued to further observe the column behaviour under extreme levels of deformation. The strain in CFRP sheets reached  $4000\mu\epsilon$  at this drift level. At 6% drift, the resistance during the first cycle was 415 kN.m, decreased to 335 kN.m during the 3<sup>rd</sup> cycle with clear bar slippage continued to be observed in the hysteretic relationship with little increase in CFRP strains, which reached  $5000\mu\epsilon$ . The test was stopped during the second cycle at 7% drift ratio.

The rotations measured at  $h$  and  $h/2$  from the column-footing interface, as well as the anchorage slip rotations are shown in Fig 3.34. It appears that the specimens with prefabricated CFRP moulds experienced less rotation compared to jacketed columns, as evidenced by this test. The data for  $\theta_h$  was recorded up to 4% drift as the LVDT stroke capacity was exceeded beyond this point, but was sufficient to make the necessary analysis. The graphs indicate that most of the rotation occurred between  $\theta_0$  and  $\theta_{h/2}$  (87%). This agreed with the low strains recorded on vertical bars at  $h/2$  compared to those at height  $h$ . At 4% drift, the rotations were measured to be 0.016 *rad*, 0.035 *rad*, and 0.040 *rad* at  $h = 0, 254$ , and 508 mm, respectively. Figs. 3.35 through 3.37 illustrate recorded strains on dowels, longitudinal bars and the FRP casing. Strain distribution on CFRP sheets along the column height is illustrated in Fig. 3.38. The specimen had flexure-dominant behaviour and failed due to lap splice slippage during the second cycle

of 5% drift, as compared to the companion retrofitted circular column (BR-CC-1) which had a drift capacity of 4%.



Rotation of the hinging region at 4% drift

### 3.2.2 LONG SQUARE COLUMN BR-SC-1

Column BR-SC-1 had a 500 mm square cross section, cast using a prefabricated CFRP stay-in-place formwork which consisted of 4 plies of CFRP sheets. What was unique about the CFRP formwork used for this column was the presence of CFRP internal ties. The casing in this case was provided with integrated ties in both cross-sectional directions to maintain the casing shape during casting and to carry the transverse tension, substituting conventional steel crossties. The column was spliced in the hinging region and the reinforcement consisted of 8 No. 25 bars. Fig 3.39 illustrates the behaviour of the specimen at different stages during testing. The lateral flexural resistance reached 464kN.m at ½% lateral drift ratio, corresponds to a load resistance of 225 kN, as shown in the hysteretic relationship depicted in Fig. 3.40. At 1% drift, the longitudinal dowel reinforcement at the column-footing interface recorded a strain of  $2200\mu\epsilon$ , indicating that the column reached flexural yield capacity of  $M_y = 510\text{kN.m}$ . The yield penetration was also recorded with the embedded portion of reinforcement showing strains reaching  $2200\mu\epsilon$ . The lateral load resistance at this stage was 246kN. When the column was

pushed to the next drift level of 2%, the flexural resistance dropped to 496kN.m and the yield capacity  $M_y$  recorded at 1% became the peak flexural resistance of the column. This wasn't a good sign for column response. The decrease in capacity was coupled with signs of pinching of the hysteresis loops and a drop in dowel strains at the column-footing interface. The recorded capacity during the first cycle at 3% drift ratio was 450kN.m, with bar slippage becoming more obvious in the hysteretic relationship. Further strength decay to 390kN.m was recorded during the third cycle, representing 76% of peak resistance. Theoretically, the column was considered to have failed at this point with further decline in load resistance at higher drift levels, with recorded capacities of 370kN.m, 310 kN.m, 288 kN.m and 278 kNm at 4%, 5%, 6%, and 7% drift ratios, respectively.

The rotations of the hinging region are illustrated in Fig. 3.41. Most of the rotation took place in the first 254 mm segment of column, with anchorage slip rotation,  $\theta_0 = 0.026$  rad and the total rotation at  $h/2$ ,  $\theta_{h/2} = 0.042$  rad at 5% drift ratio. Total rotation at height  $h$ ,  $\theta_h$  was measured to be 0.05 rad. The anchorage slip rotation showed a drastic increase from 0.007 rad at ½% drift ratio to 0.026rad at 2% drift ratio. This was consistent with the observed deterioration in column capacity beyond 2% drift.

The recorded strains on dowels and longitudinal steel are illustrated in Fig 3.42 & 3.43. Dowels yielded during the first cycle at 1% drift with recorded strain between  $2000\mu\epsilon$  and  $3000\mu\epsilon$ . The maximum strain recorded in longitudinal reinforcement was  $1000\mu\epsilon$  at 1% drift, dropped to  $500\mu\epsilon$  at 2% drift and the graph became horizontal beyond that indicating lap splice slippage.

The use of internal FRP crossties was one of the parameters that were investigated experimentally. The crossties were expected to affect the degree of confinement to be provided by the casing, as well as the stability of compression bars against buckling. It was decided to use a single layer of crossties in BR-SC-1.

Maximum strains recorded on CFRP sheet were at the bottom portion of the casing, reaching  $3000\mu\epsilon$  as shown in Fig 3.44. This level of strain was attributed to lateral expansion of damaged concrete inside the casing causing the column to bulge out as illustrated in Fig. 3.39

Fig 3.45 shows the recorded strains on CFRP ties during testing. Strain gauge No. 15, installed on the first tie from the bottom, reached  $1100\mu\epsilon$  and went off-scale during the first cycle at 2% drift. However, the other end of the tie (strain gauge No. 21) indicated higher strains at 1% drift reaching  $1800\mu\epsilon$  and approaching  $2500\mu\epsilon$  during the first cycle at 2% drift. The first tie ruptured at this stage, even though the recorded strains were considerably lower than the rupturing strain. It was speculated that the wires of the strain gauge could have been damaged due to damaged concrete and stopped functioning prematurely. The Strain Gauge No. 20 attached to the tie in the second row recorded  $2000\mu\epsilon$  and reached  $3000\mu\epsilon$  during the first cycle of 2% drift. This strain gauge also went off-scale and probably for the same reason mentioned above. Unfortunately, the strain readings on these two rows of ties did not provide the required information to explain the tie behaviour. However, the Gauge No. 18 attached to the forth row of ties at 550 mm above the column-footing interface (Fig 3.45) provided valuable data. The data showed that this tie had developed a strain of  $12000\mu\epsilon$  during the first cycle at  $\frac{1}{2}\%$  drift, dropped slightly before increasing again to  $16000\mu\epsilon$  and going off-scale. This means that the tie was able to develop stresses up to the failure stress. Except for some increase in the CFRP casing strains, there were no signs of improvement on the other parameters.

To further examine the behaviour of Column BR-SC-1, a portion of the casing was removed to investigate the state of concrete and reinforcement inside. It can be seen in Fig 3.39 that the longitudinal bar buckled causing debonding and consequent loss in strength within the splice region. Therefore, the number of crossties was increased in the next specimen, while also increasing the splice length of bars in Column BR-CS-2. The specimen failed during the 3<sup>rd</sup> cycle at 3% drift due to the buckling and consequent slippage of longitudinal reinforcement within the splice region. The drift capacity of the specimen was 2% and a square casing with a single layer of crossties of the size provided



was not sufficient to maintain sufficiently high drift capacity in columns with spliced reinforcement based on the old practice of using 20 times the bar diameter as splice length.

### **3.2.3 LONG SQUARE COLUMN BR-SC-2**

Column BR-SC-2 was designed and built following the test of specimen BR-SC-1, which showed poor performance, as described in 3.2.2. While the number of CFRP plies used to prepare the casing was similar to BR-SC-1, i.e. 4 plies, double cross ties were used instead of single ties. In addition, the current practice of using longer starter bars (60-bar diameter) was implemented, compared to the 20-bar diameter used in BR-SC-1 (representing pre-1970s codes). Fig 3.47 illustrates the column behavior at selected drift levels while the lateral Force-Drift and Moment-Drift relationships are shown in Fig 3.48.

The maximum-recorded moment at  $\frac{1}{2}\%$  drift was 475 kN.m, corresponding to lateral force of 230 kN. The dowels reached 2000  $\mu\epsilon$  at the column-footing interface. At 1% drift, the moment resistance reached 570 kN.m. Reinforcing steel yielded at this stage and entered into strain hardening as recorded by strain gauges 2 and 8, after which both gauges went off-scale (Fig. 3.50). The moment capacity continued to increase at higher levels of deformations, reaching 640 kN.m and 650 kN.m at 2% and 3% drift, respectively. At 3% drift, the hysteretic graph showed signs of pinching although very little. The load capacity at 4% to 6% drift indicates superior performance as the moment capacity was maintained until approximately 640 kN.m in the push direction and 690 kN.m in the pull direction. The minor pinching in the graphs indicates very little or no slippage of reinforcement within the splice region. This is evident by the close strain readings recorded in both the starter bars and the vertical reinforcement. Fig 3.50 shows dowel strains, were longitudinal reinforcement strains along the plastic hinge region are illustrated in Fig 3.51.

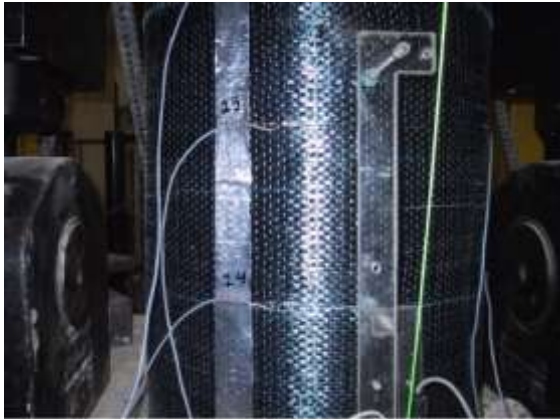
The rotations measured at  $h$  and  $h/2$  from the column-footing interface are shown in Fig.3.49. Most of the rotation occurred at  $h/2$ , as the values for  $\theta_{h/2}$  and  $\theta_h$  were very close. This observation is consistent with the strain values recorded on longitudinal

reinforcement, as little increase took place in recorded strains at height  $h$ , relative to those recorded at  $h/2$ . The anchorage slip rotations were relatively small. This is due to the large splice length providing sufficient bond. Rotations at 5% drift were measured as 0.018 rad., 0.04 rad., and 0.0425 rad at  $h = 0, 250$  and  $500$  mm, respectively.

Strain gauge locations on CFRP sheets and cross ties are shown in Figs. 3.52 and 3.53 along with recorded values, respectively. Graphs given in these figures indicate that no significant strain values were observed on CFRP sheets, with maximum strain at the bottom portion of the casing reaching  $1500\mu\epsilon$ . This level of strain indicates little damage to concrete inside the casing, which is evident by the minor bulging of the casing observed in the outer surface of the column (Fig 3.47).

Fig 3.53 shows recorded strains on CFRP ties during testing. Highest stresses in ties were recorded near the bottom of the column, compared to BR-SC-1, which had high strains at distance  $h$  above the base. Strain gauge No. 13, installed on the first tie from the bottom, experienced low strains during the first two cycles before developing a strain of  $16000\mu\epsilon$  at 2% drift and going off-scale. The tie ruptured at this stage as seen in Fig 3.55 (f). Other ties continued to show strain readings though very little with maximum reading being  $300\mu\epsilon$ . It is believed that the use of two cross ties along with longer starter bars enhanced the degree of confinement provided by the casing, as well as the stability of compression bars against buckling. This is evident by the superior performance of the column. Strain distribution on CFRP sheets along the column height is illustrated in Fig.3.54.

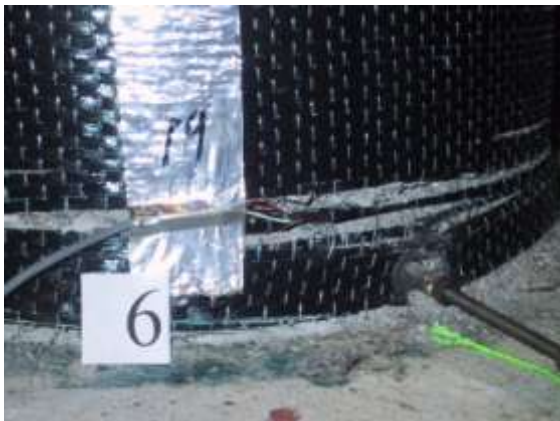
In conclusion, the specimen behaved very well under cycling loading, and sustained large drift levels while maintaining its strength. The column failed during the first cycle of 7% drift ratio due to the rupturing of longitudinal reinforcement (Fig 3.53 a and b). The drift capacity of the specimen was considered to be 6%.



(a) 4% Drift

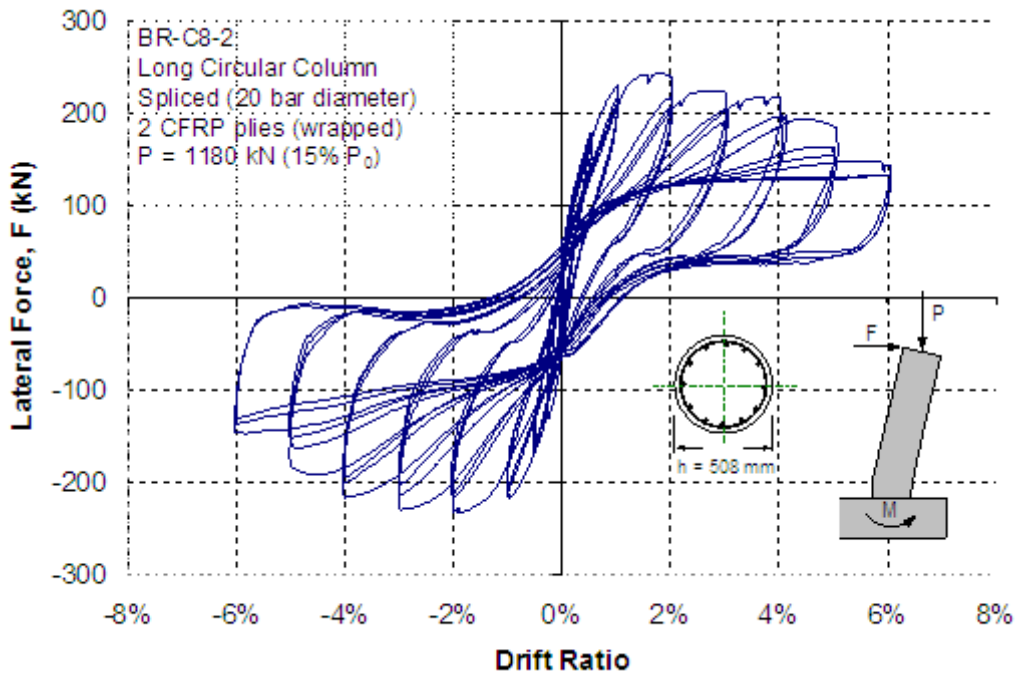


(b) 5% Drift

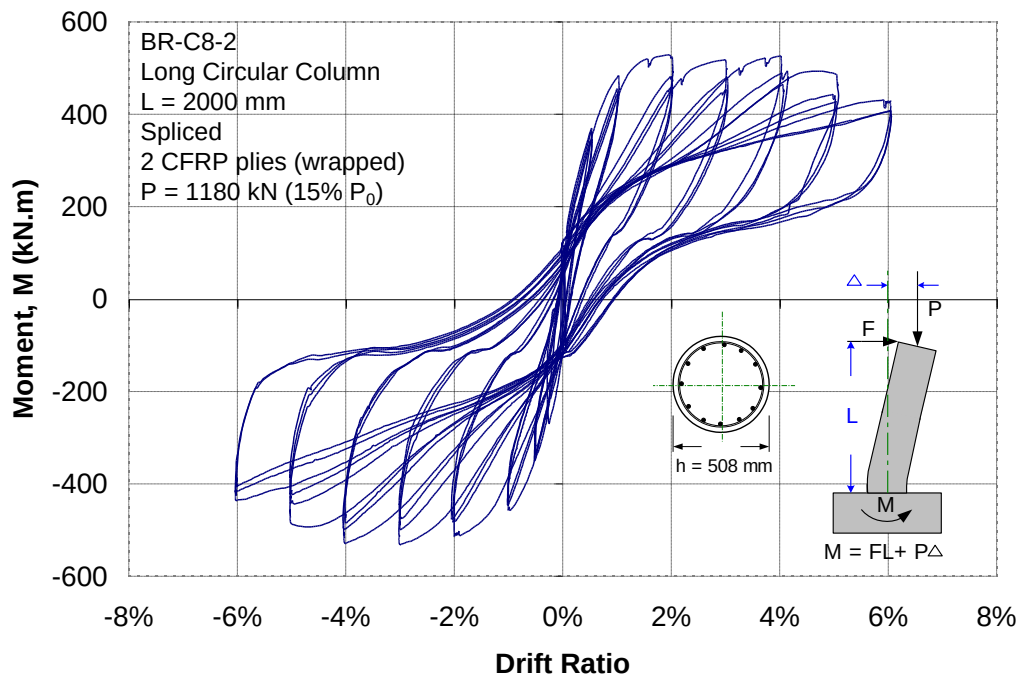


(c) 5% Drift

**Fig. 3.1** Behavior of column BR-C8-2 at different drift levels

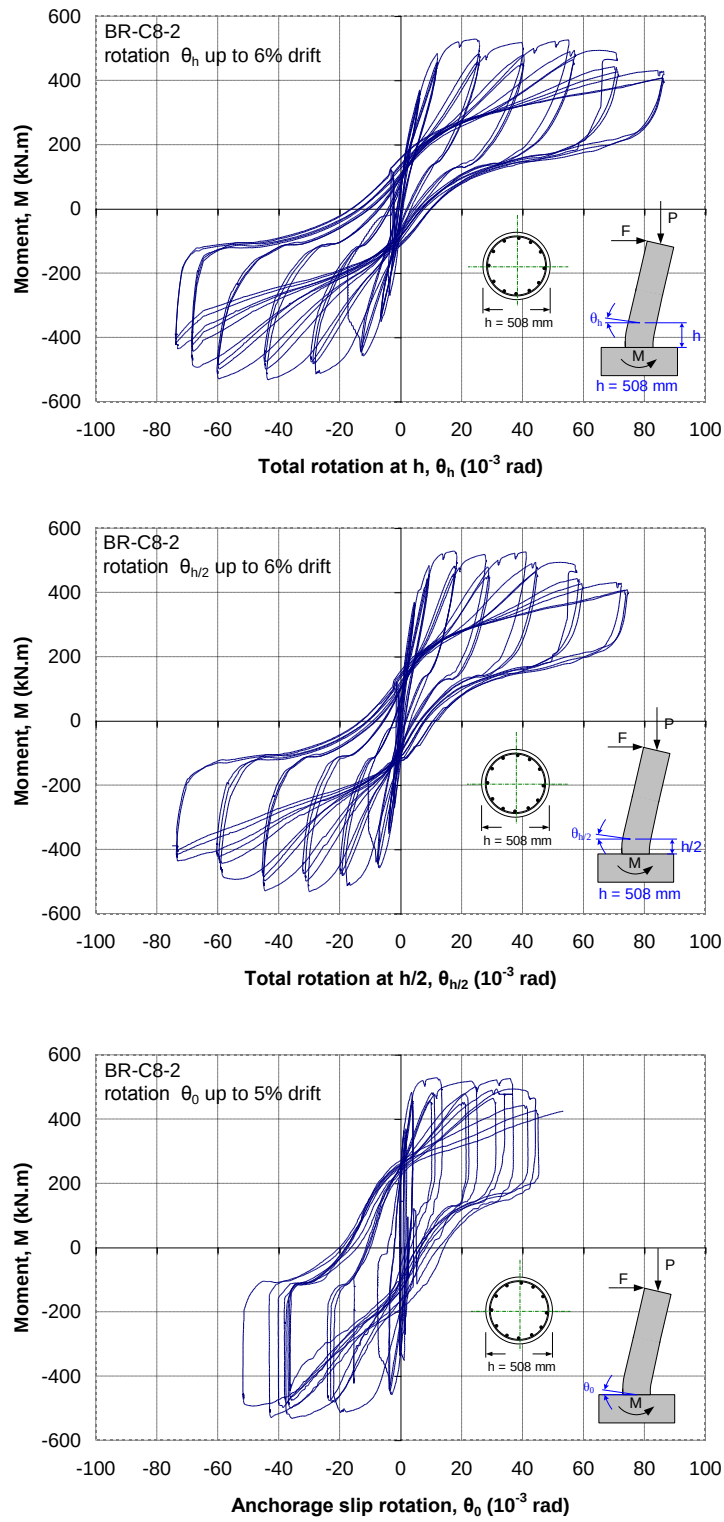


(a) Force-Drift relationship

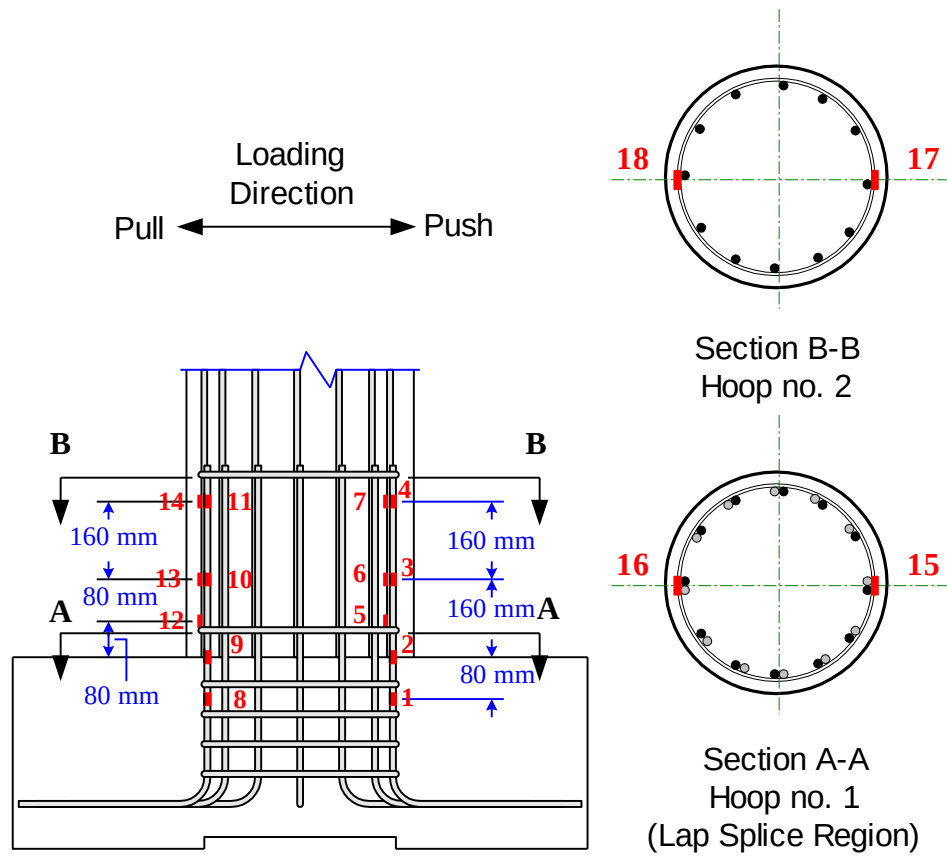


(b) Moment-Drift relationship

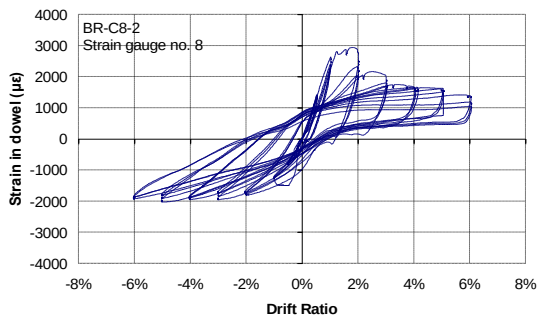
Fig. 3.2 Hysteretic relationships for column BR-C8-2



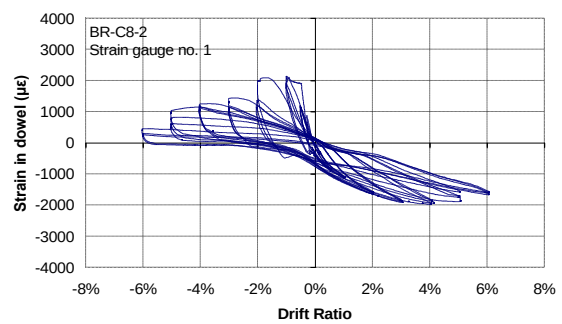
**Fig. 3.3** Rotations of the hinging region for column BR-C8-2



Strain gauge locations in reinforcement steel

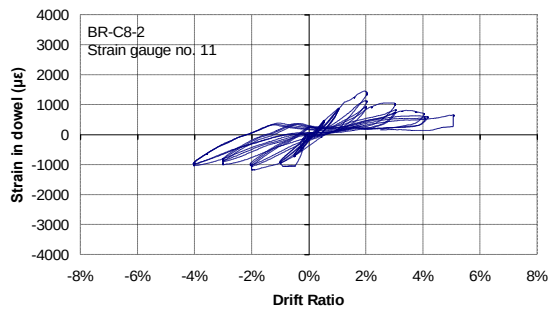


Strain gauge no. 8

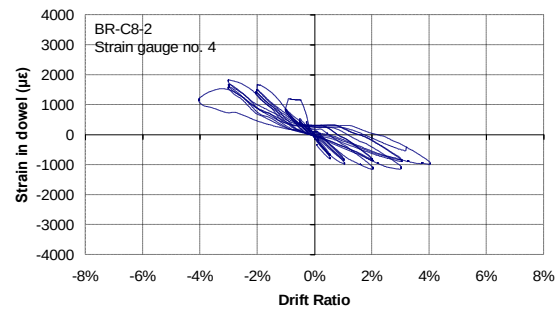


Strain gauge no. 1

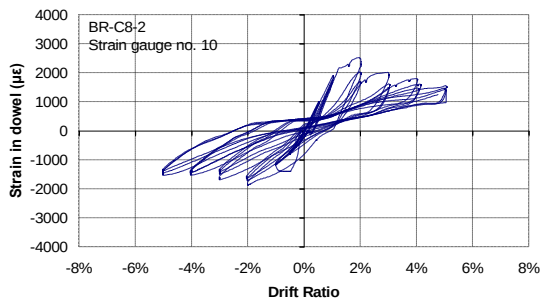
**Fig. 3.4** Dowel strains for column BR-C8-2



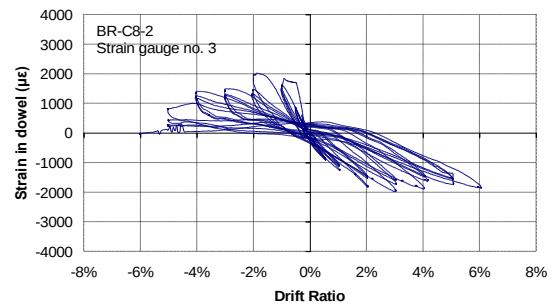
Strain gauge no. 11



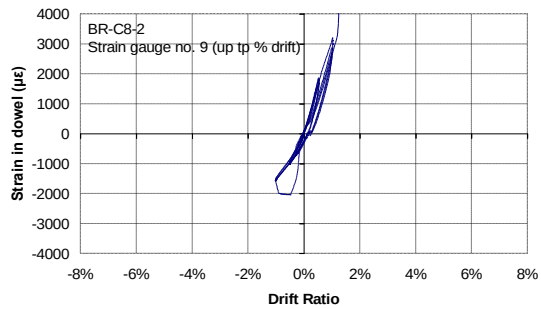
Strain gauge no. 4



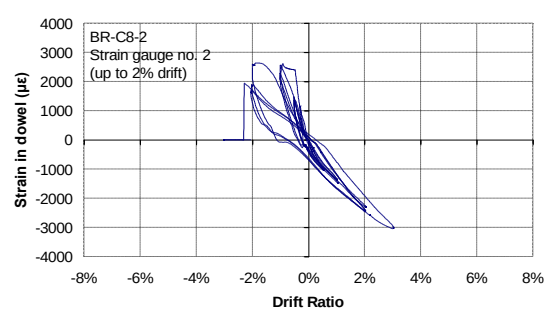
Strain gauge no. 10



Strain gauge no. 3



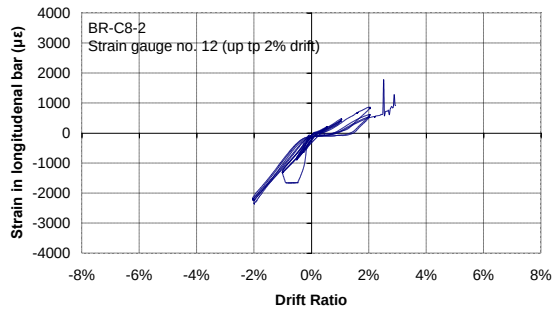
Strain gauge no. 9



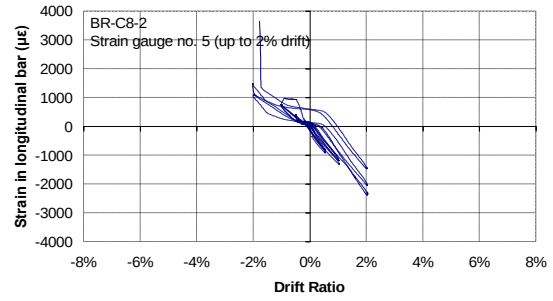
Strain gauge no. 2

**Fig. 3.4 (Cont'd) Dowel strains for column BR-C8-2**

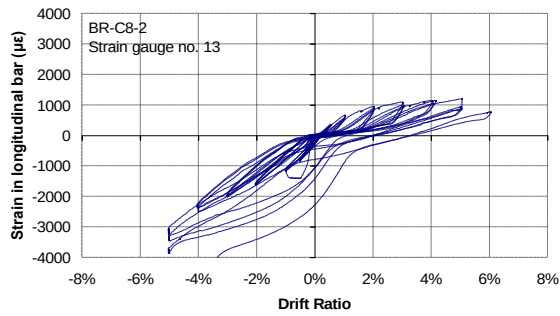




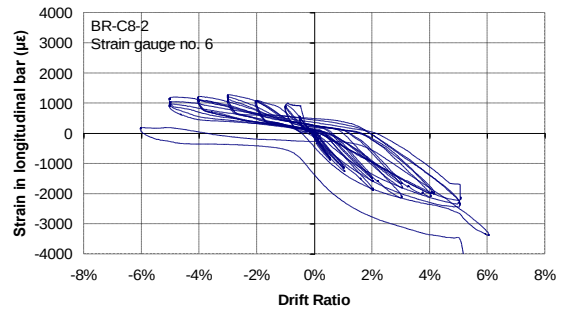
Strain gauge no. 12



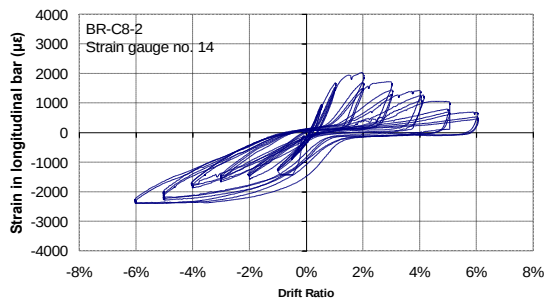
Strain gauge no. 5



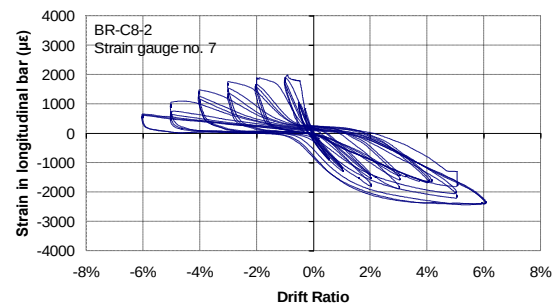
Strain gauge no. 13



Strain gauge no. 6



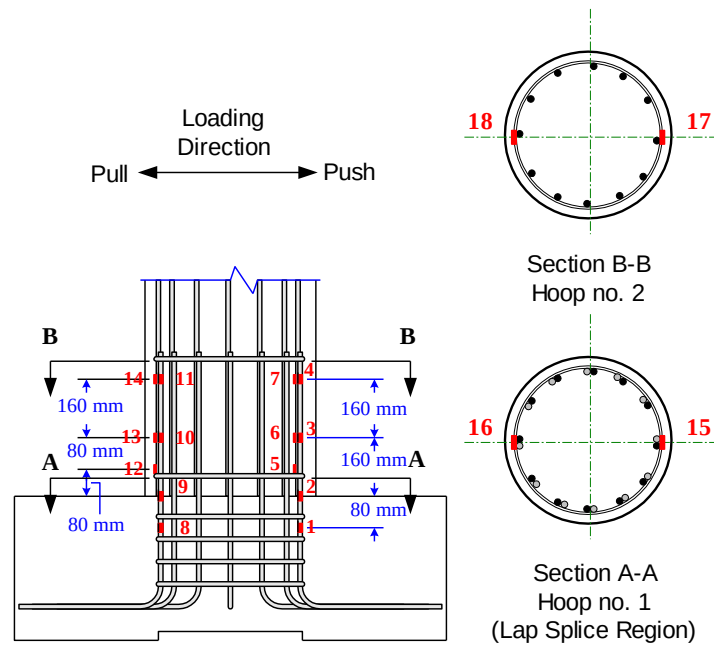
Strain gauge no. 14



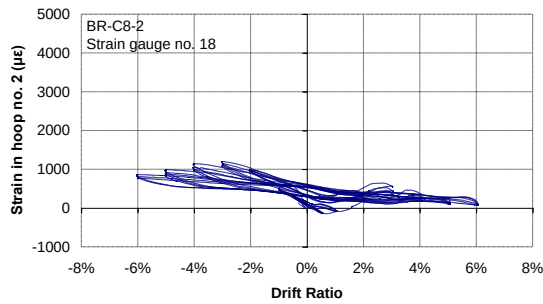
Strain gauge no. 7

**Fig. 3.5** Longitudinal reinforcement steel strains for column BR-C8-2

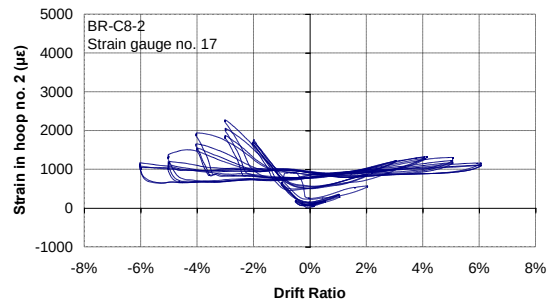




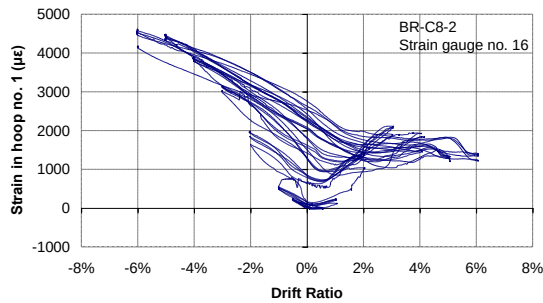
Strain gauge locations in reinforcement steel



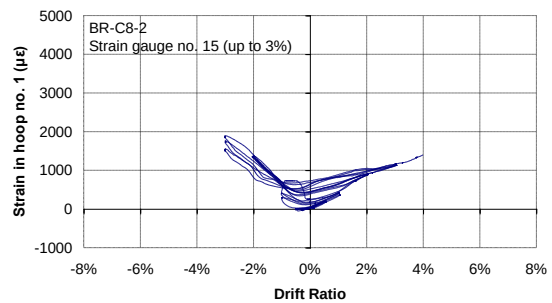
Strain gauge no. 18



Strain gauge no. 17

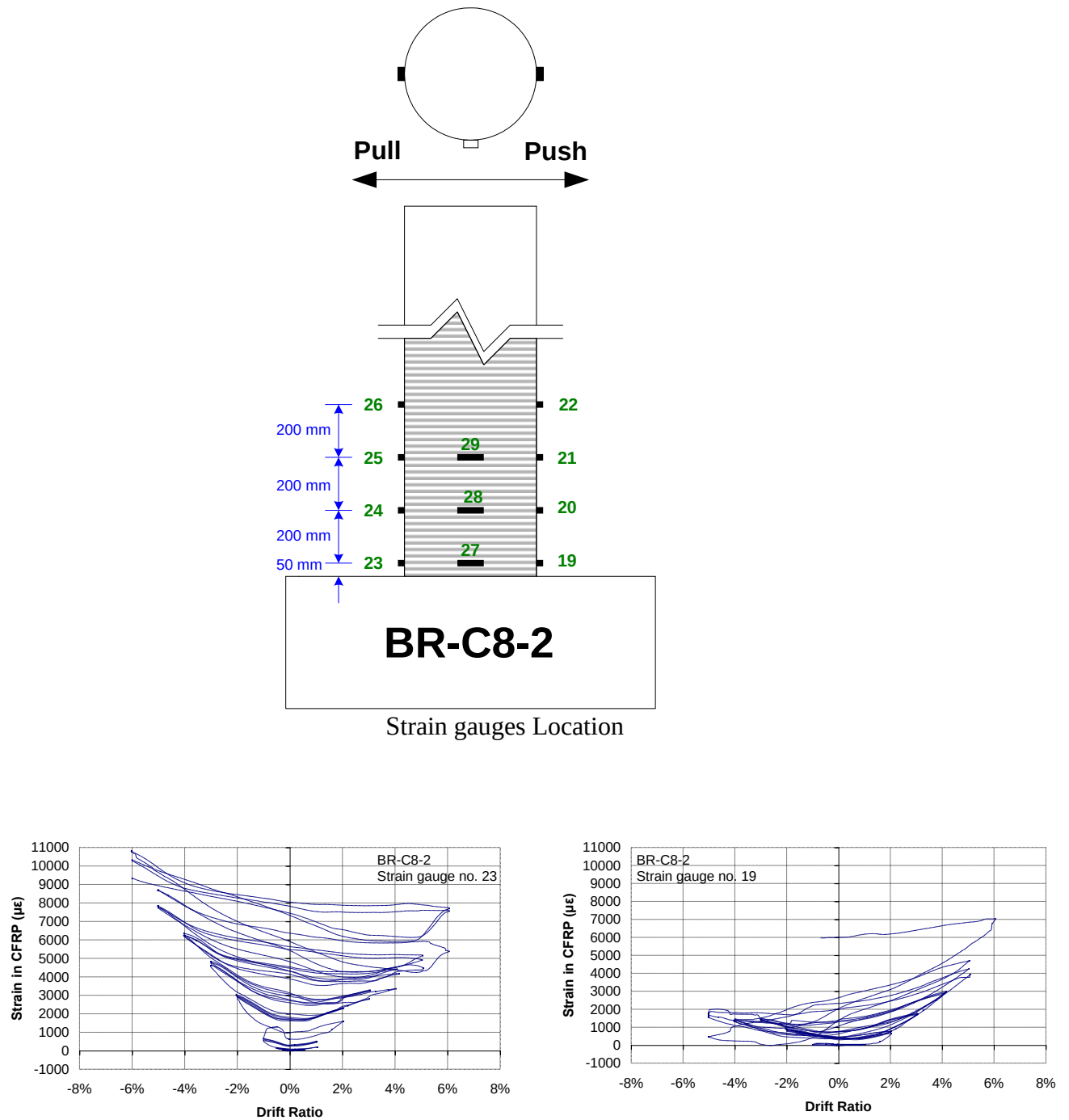


Strain gauge no. 16

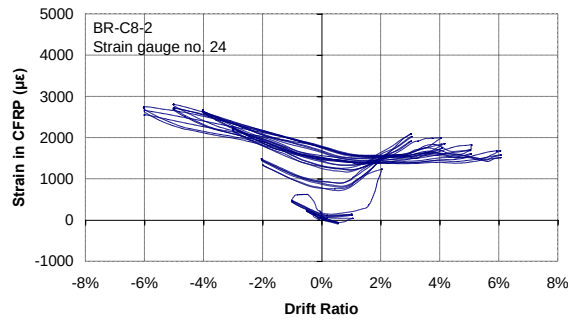


Strain gauge no. 15

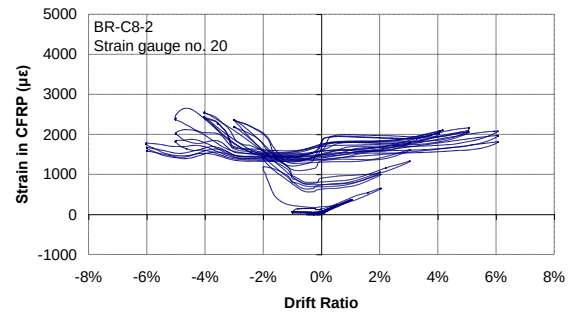
**Fig. 3.6** Transverse reinforcement steel strains for column BR-C8-2



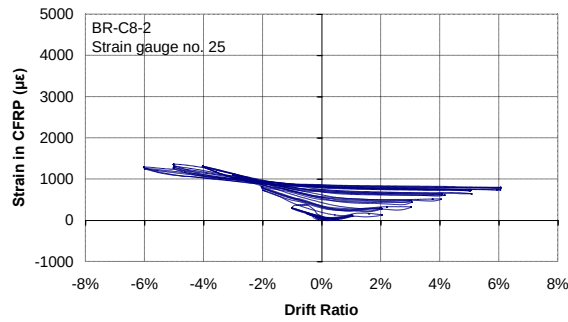
**Fig. 3.7** CFRP sheet strains for column BR-C8-2



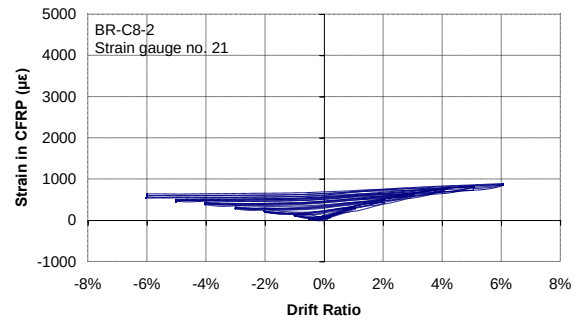
Strain gauge no. 24



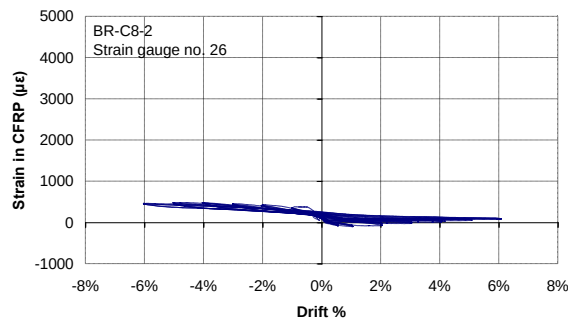
Strain gauge no. 20



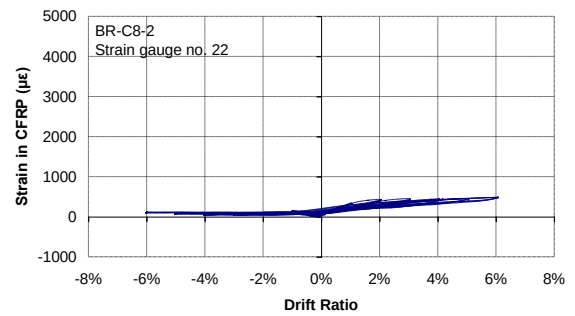
Strain gauge no. 25



Strain gauge no. 21

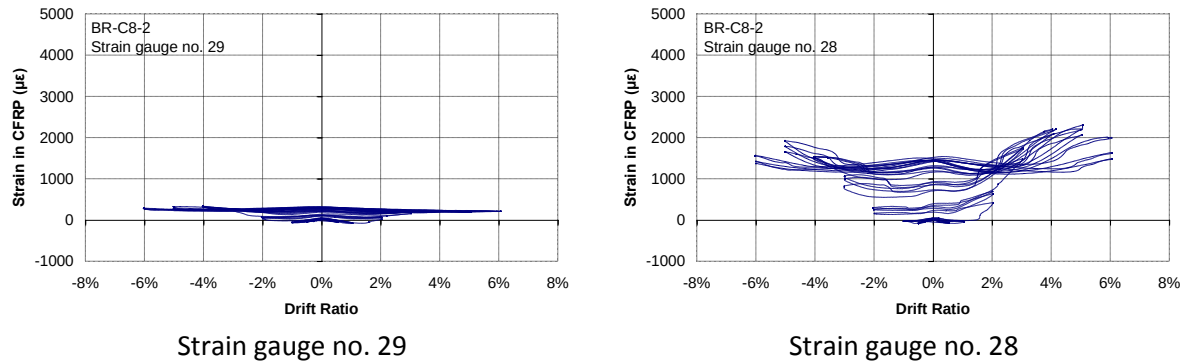


Strain gauge no. 26

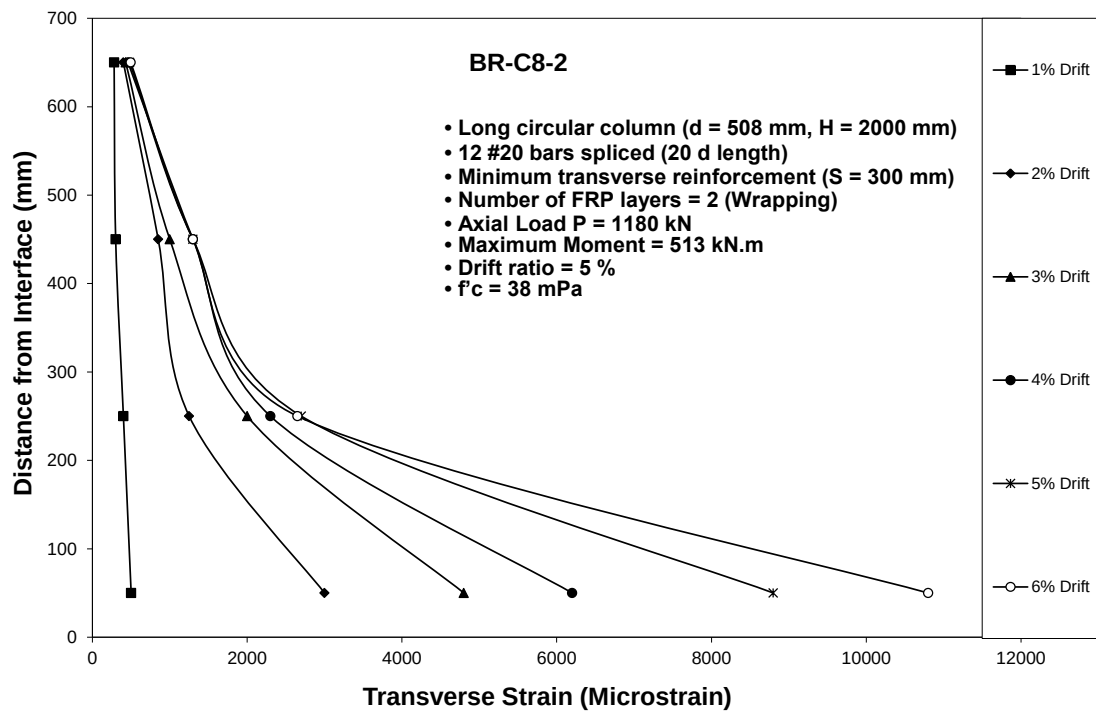


Strain gauge no. 22

**Fig. 3.7(Cont'd)** CFRP sheet strains for column BR-C8-2



**Fig. 3.7(Cont'd)** CFRP sheet strains for column BR-C8-2



**Fig. 3.8** Fiber strain distribution along column BR-C8-2



(a) 2% drift

**Fig. 3.9** Behavior of column BR-C10-2 at different drift levels



(b) 4% Drift

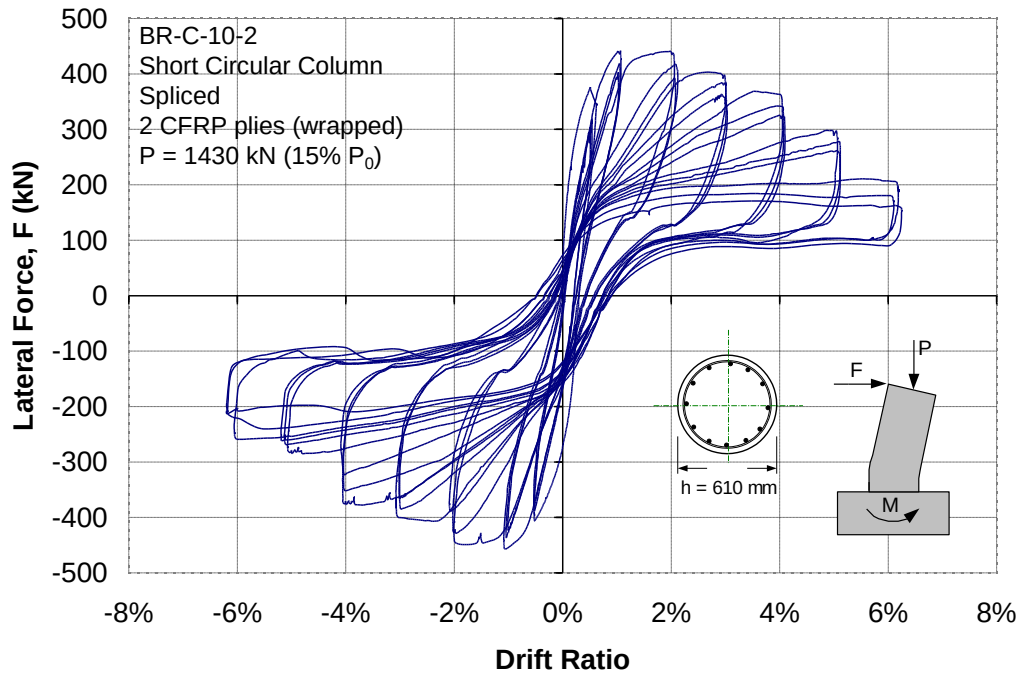


(c) 5% Drift

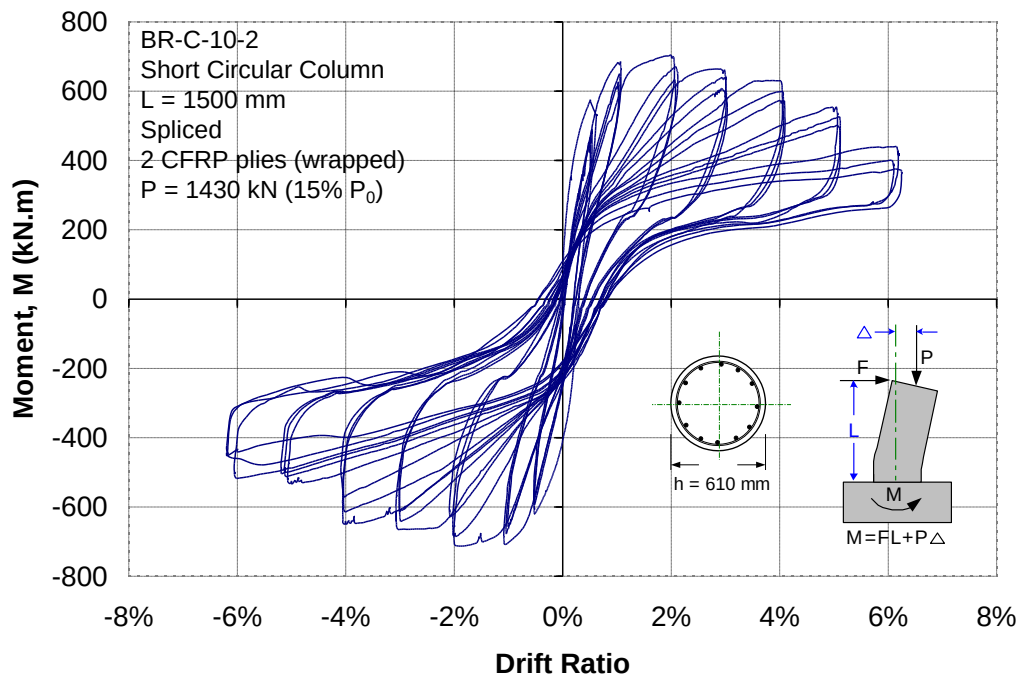


(d) 6% Drift

**Fig. 3.9(Cont'd)** Behavior of column BR-C10-2 at different drift levels



(a) Force-drift relationship



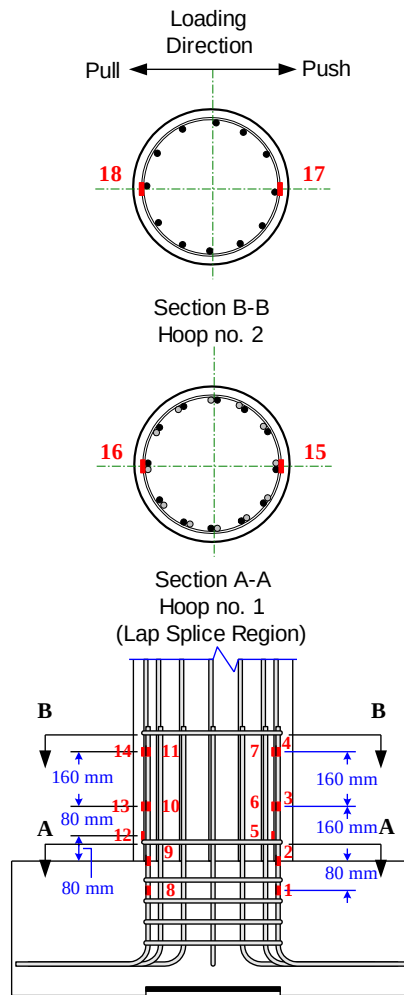
(b) Moment-drift relationship

**Fig. 3.10** Hysteretic relationship for column BR-C10-2

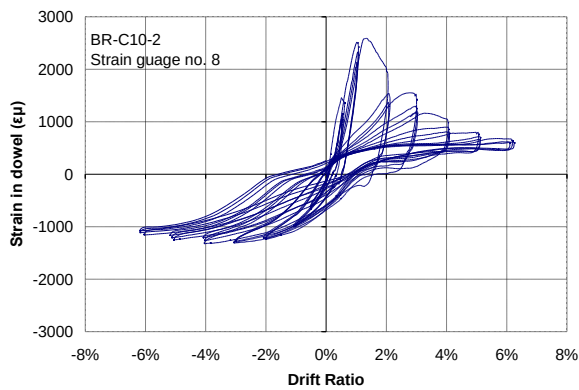




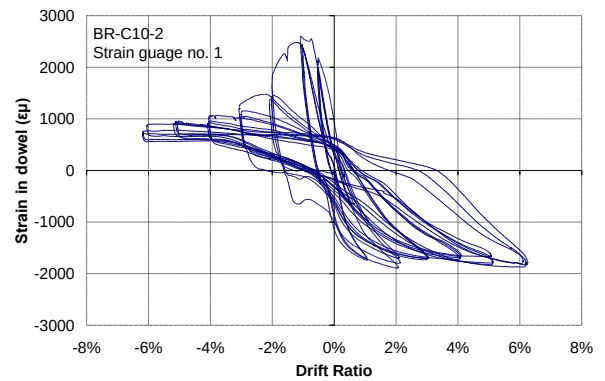




Strain gauges locations

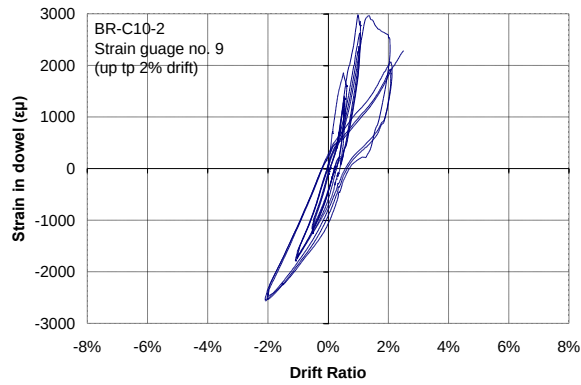


Strain gauge no. 8

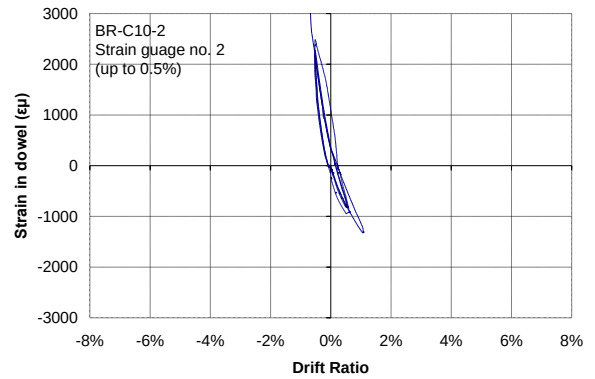


Strain gauge no. 1

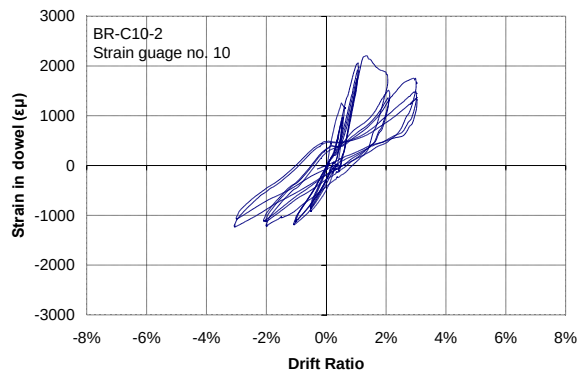
**Fig. 3.12** Dowel strains for column BR-C-10-2



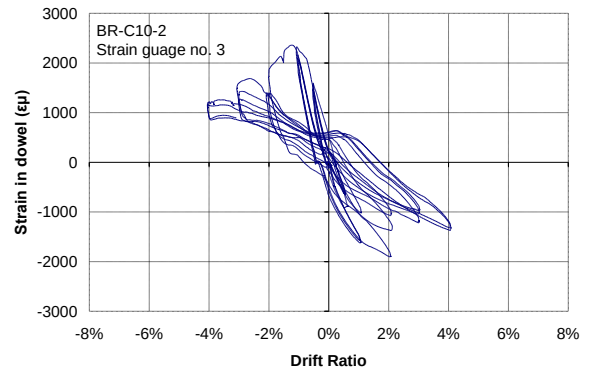
Strain gauge no. 9



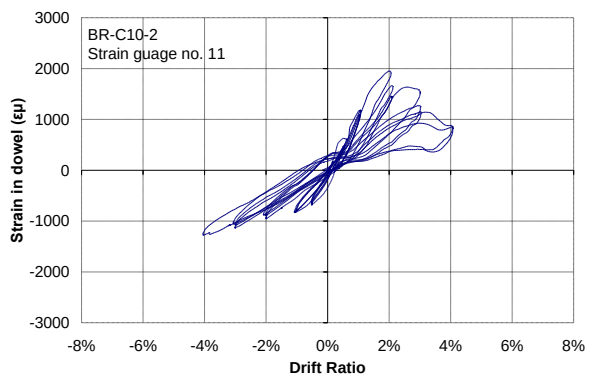
Strain gauge no. 2



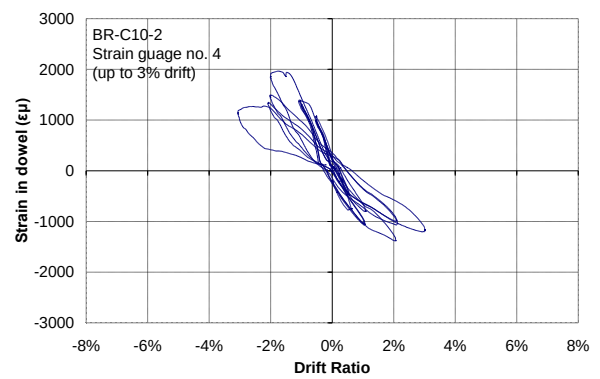
Strain gauge no. 10



Strain gauge no. 3

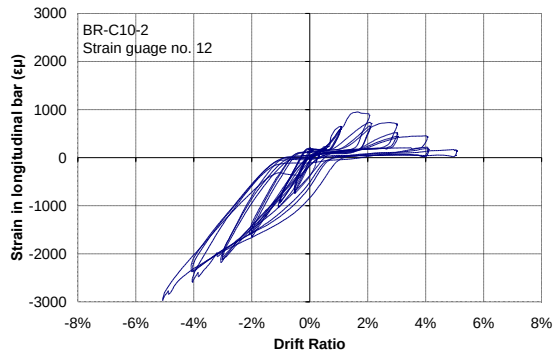


Strain gauge no. 11

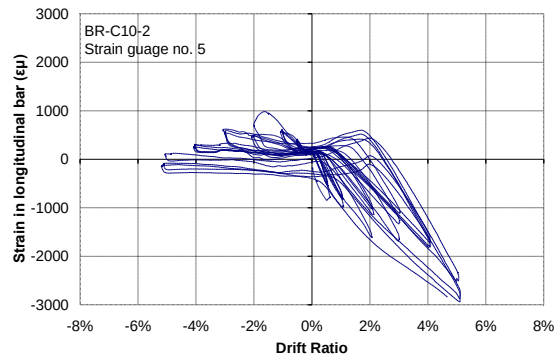


Strain gauge no. 4

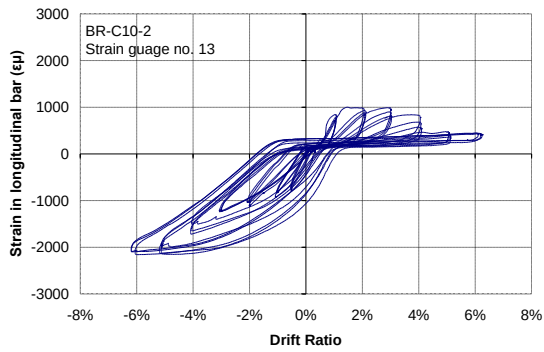
**Fig. 3.12(Cont'd)** Dowel strains for column BR-C-10-2



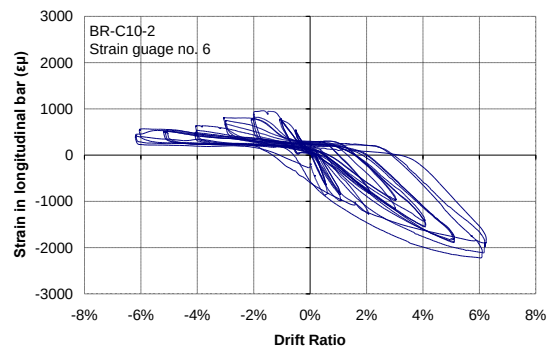
Strain gauge no. 12



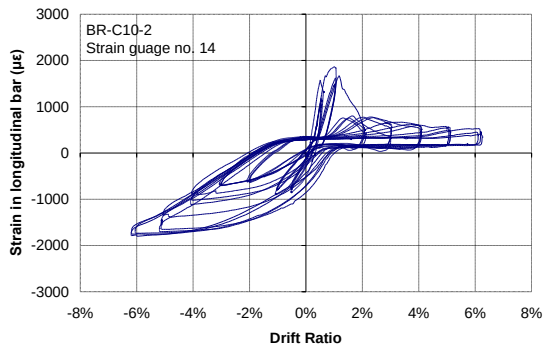
Strain gauge no. 5



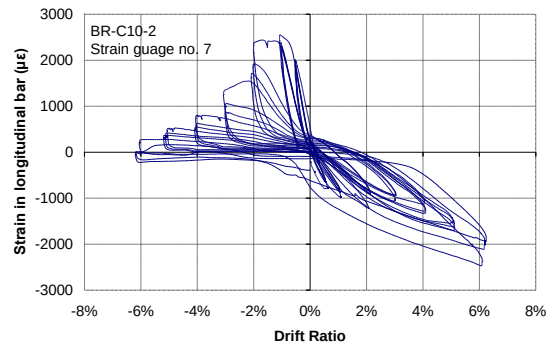
Strain gauge no. 13



Strain gauge no. 6

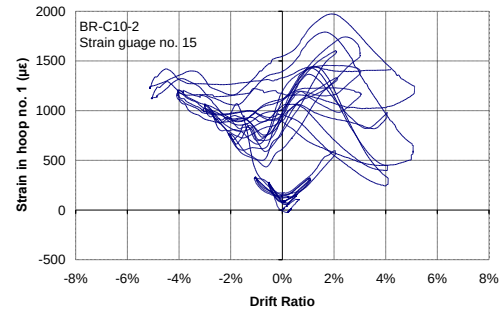
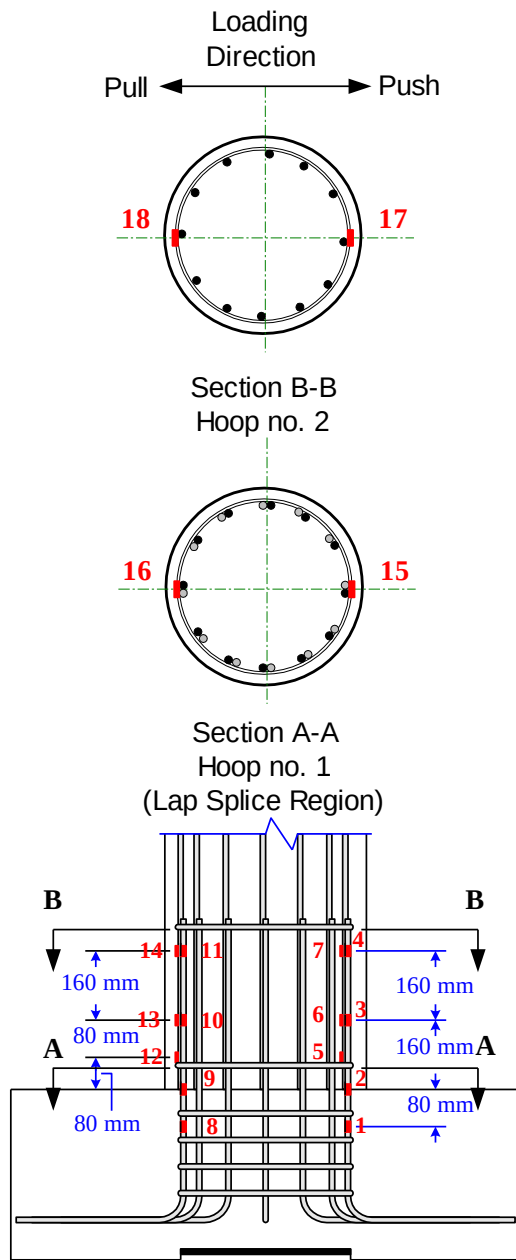


Strain gauge no. 14

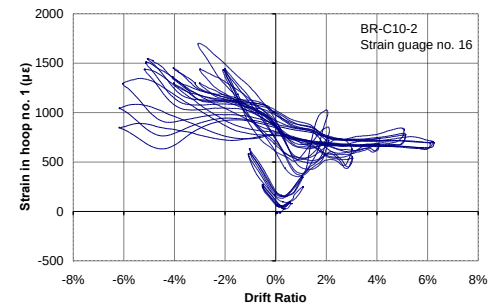


Strain gauge no. 7

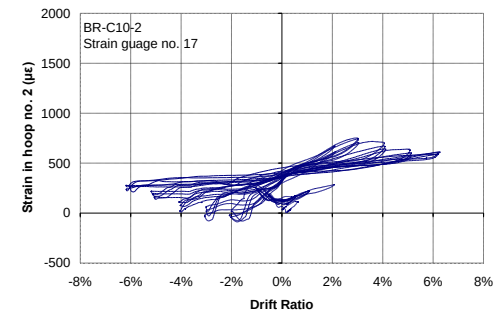
**Fig. 3.13** Longitudinal reinforcement steel strains for column BR-C10-2



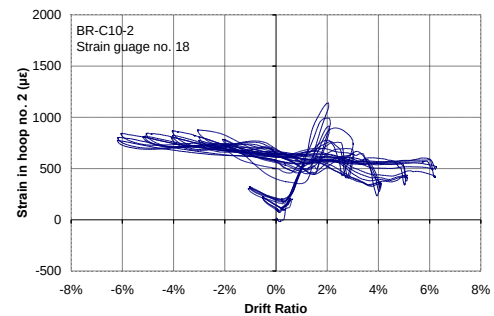
Strain gauge no. 15



Strain gauge no. 16

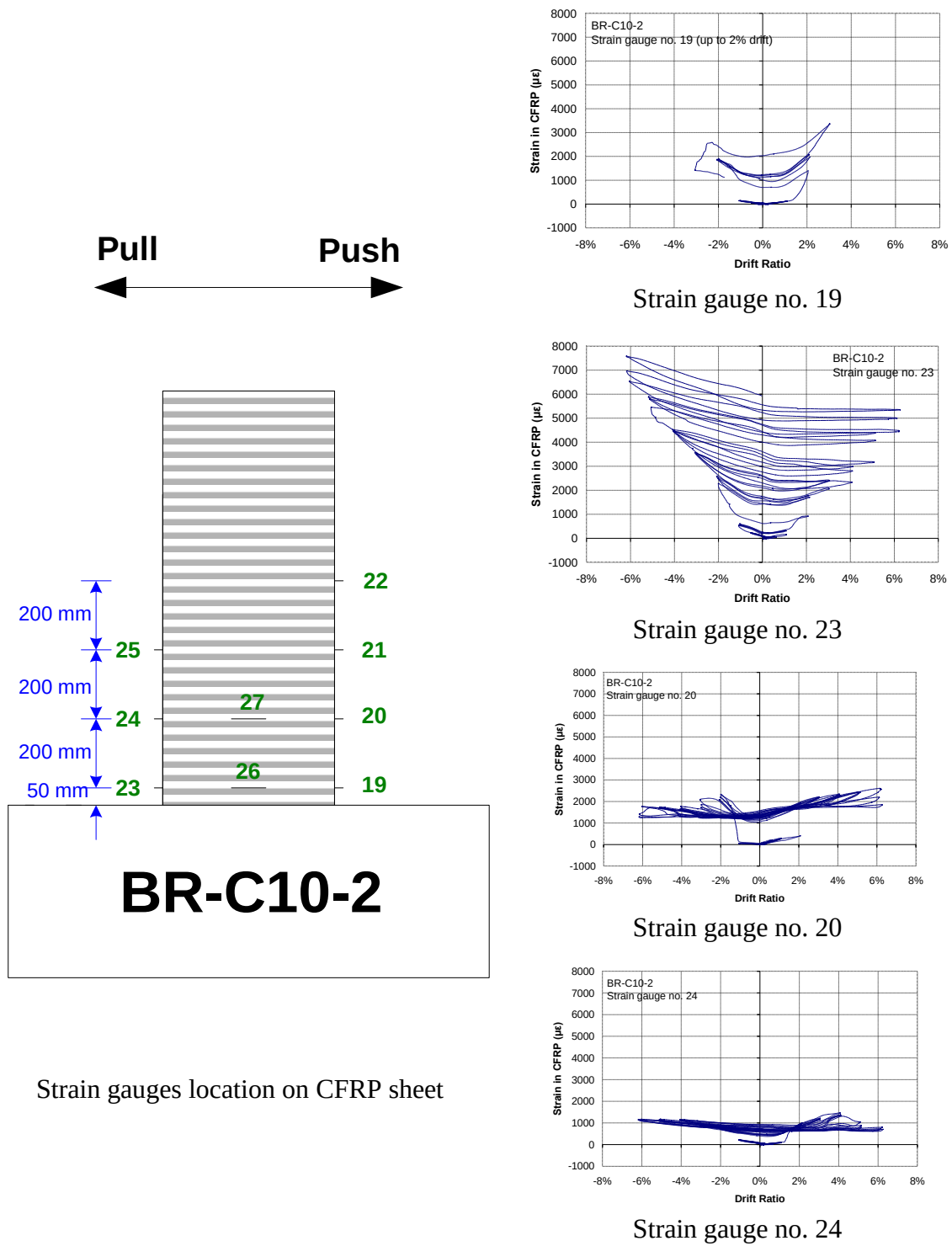


Strain gauge no. 17

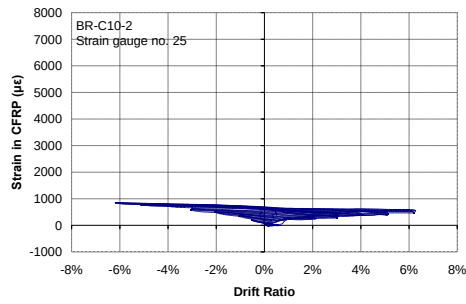


Strain gauge no. 18

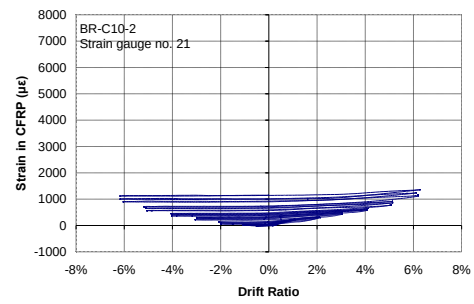
**Fig. 3.14** Transverse reinforcement steel strains for column BR-C10-2



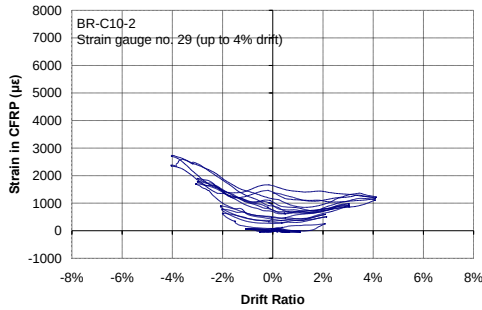
**Fig. 3.15** CFRP sheet strains for column BR-C10-2



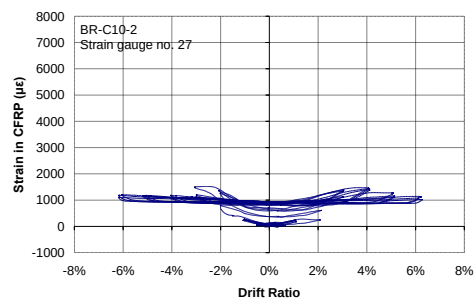
Strain gauge no. 25



Strain gauge no. 21



Strain gauge no. 26



Strain gauge no. 27

Fig. 3.15(Cont'd) CFRP sheet strains for column BR-C10-2

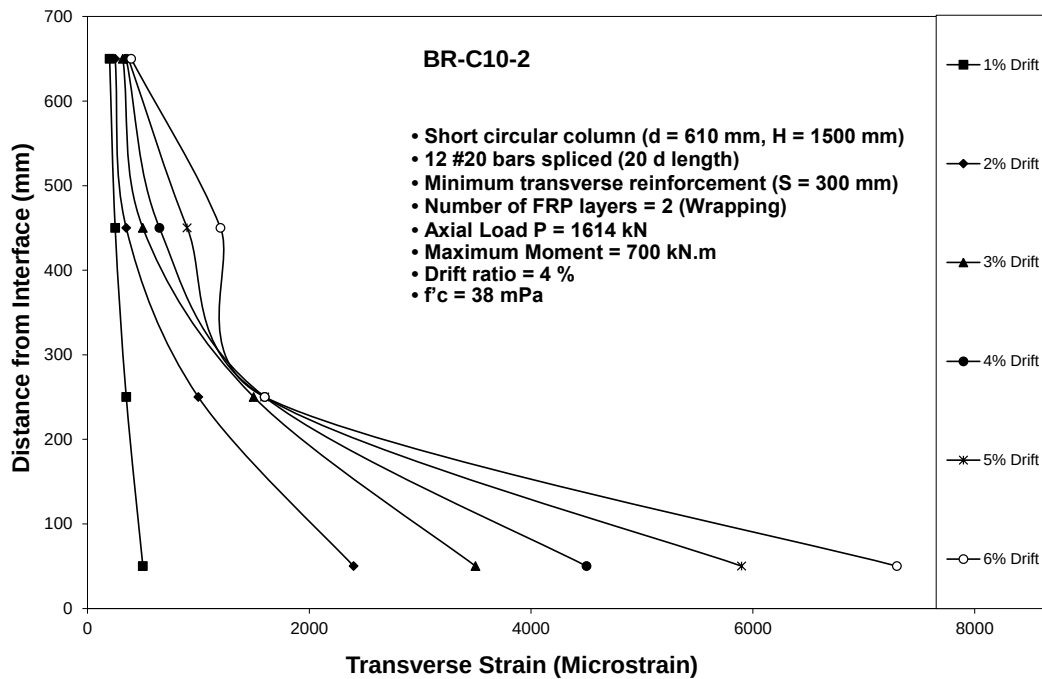


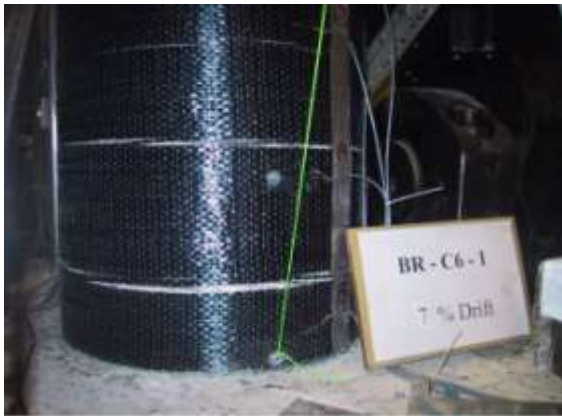
Fig. 3.16 Fiber strain distribution along column BR-C10-2



**(a)** 5% Drift



**(b)** 6% Drift



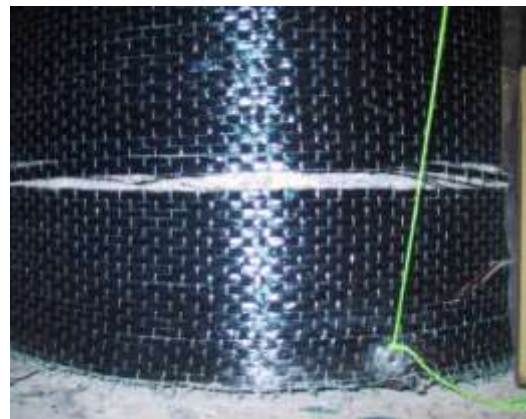
**(c)** 7% Drift



**(d)** 7.5 % Drift

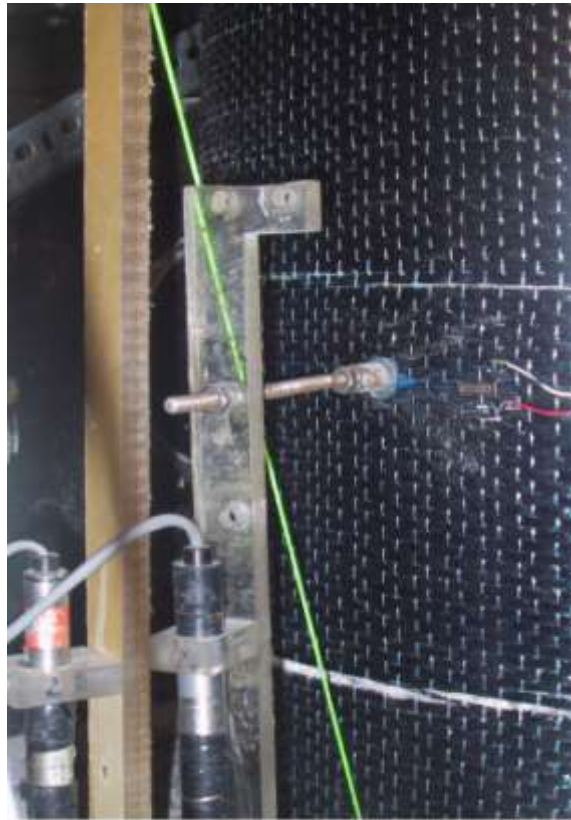


**(e)** 8% Drift



**Fig. 3.17** Behavior of column BR-C6-1 at different drift levels

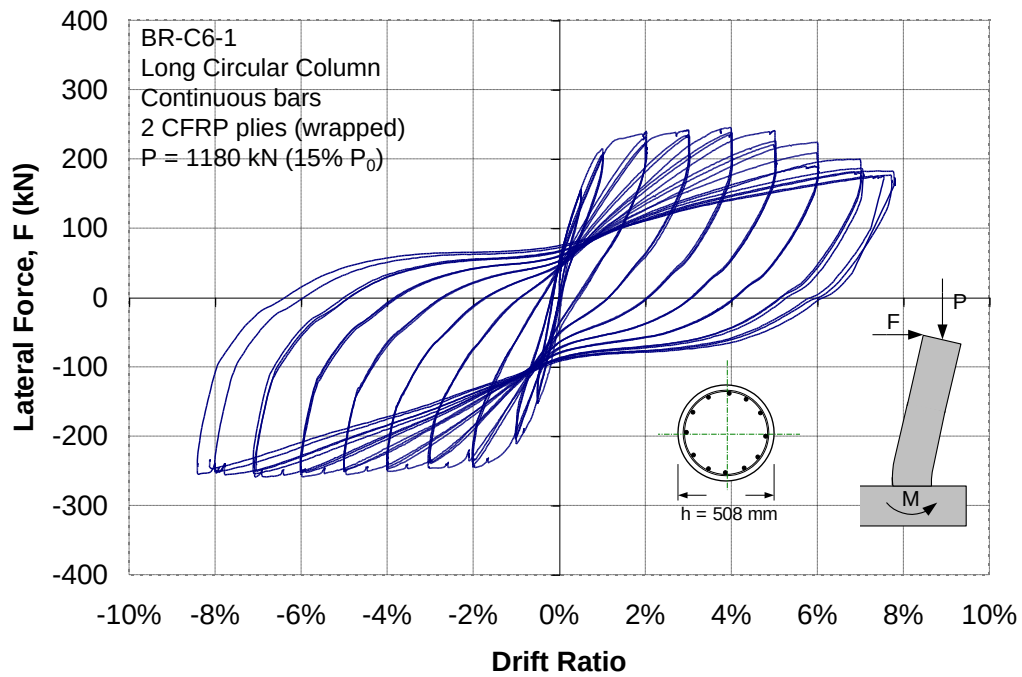




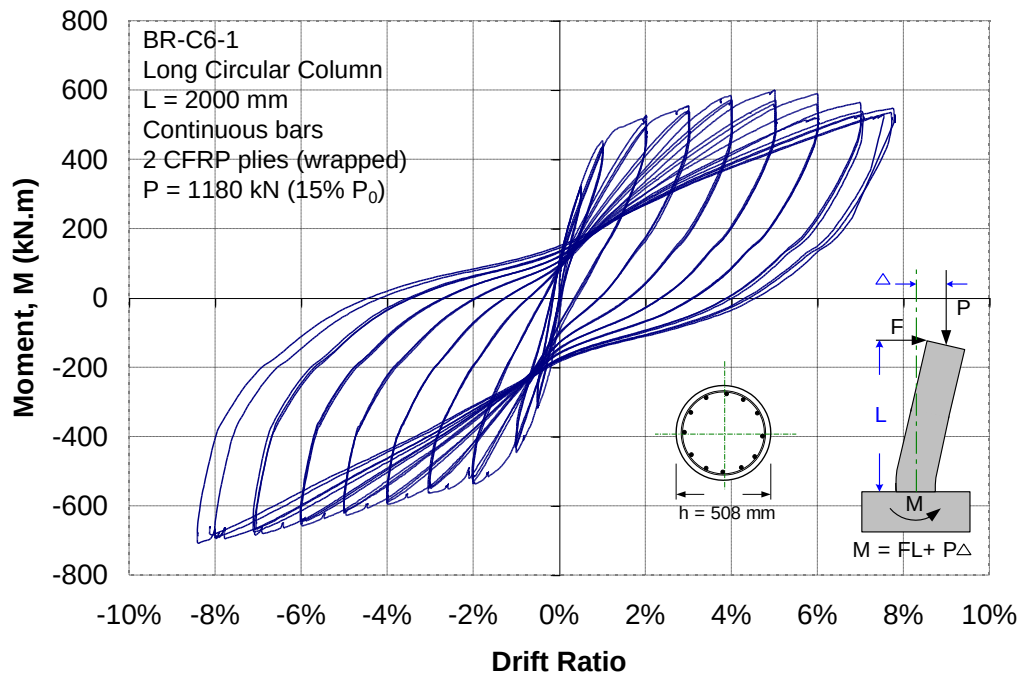
(f) At the end of test

**Fig. 3.17(Cont'd)** Behavior of column BR-C6-1 at different drift levels



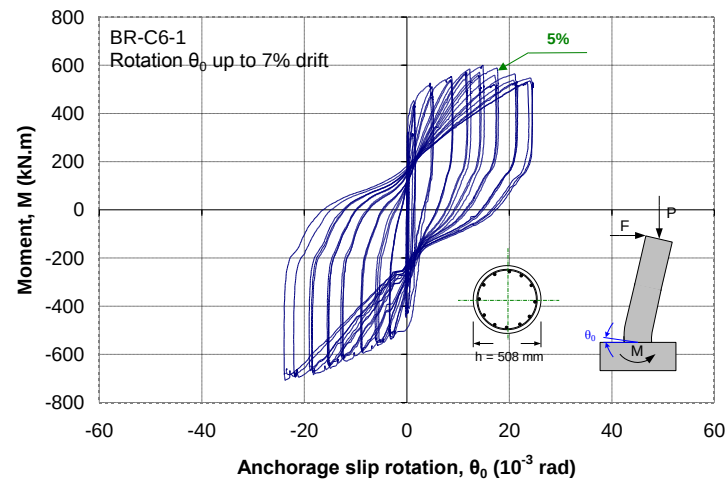
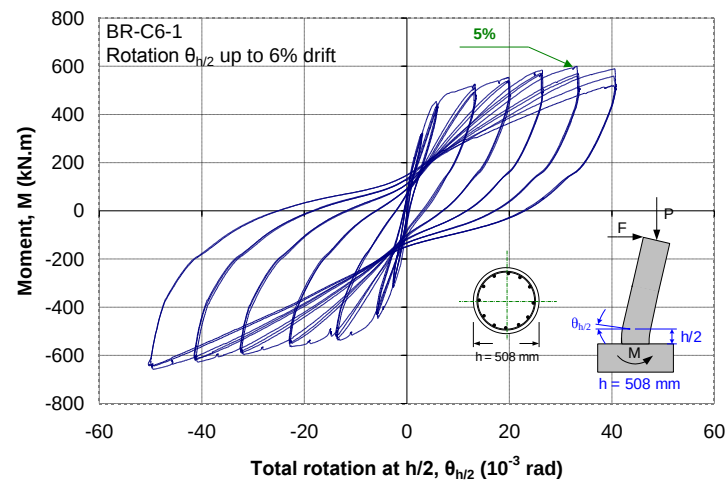
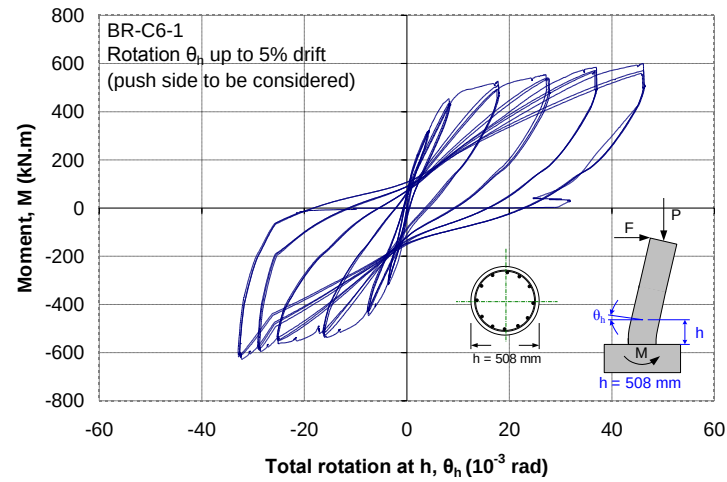


a) Force-Drift relationship

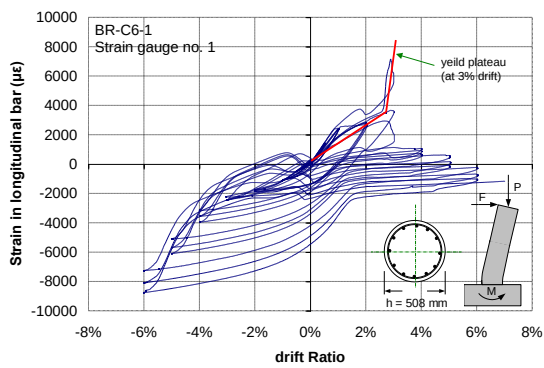
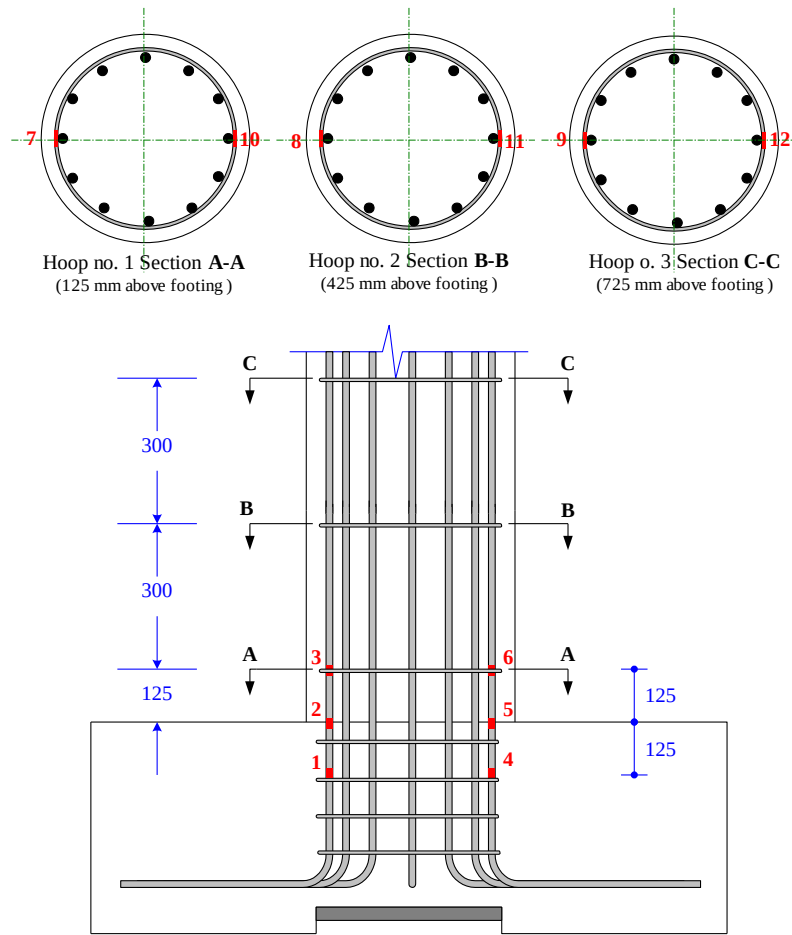


b) Moment-Drift relationship

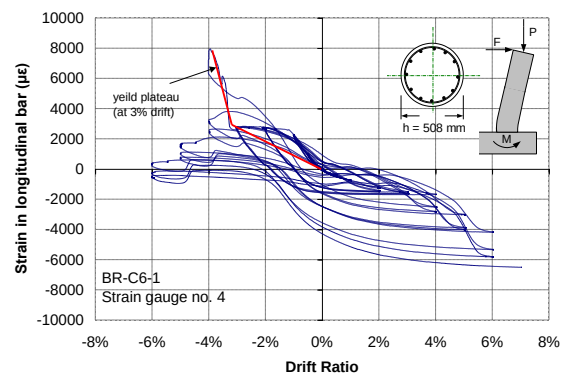
**Fig. 3.18** Hysteretic relationships for column BR-C6-1



**Fig. 3.19** Rotation of the hinging region for column BR-C6-1

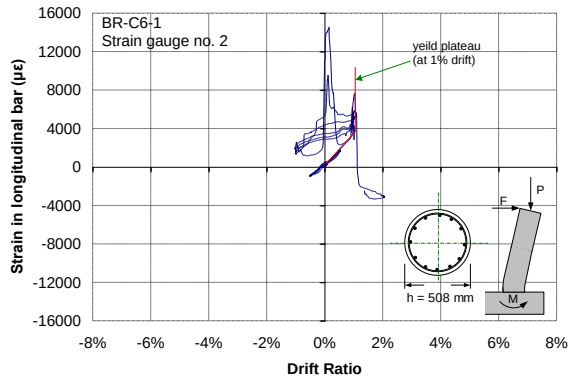


Strain gauge no. 1

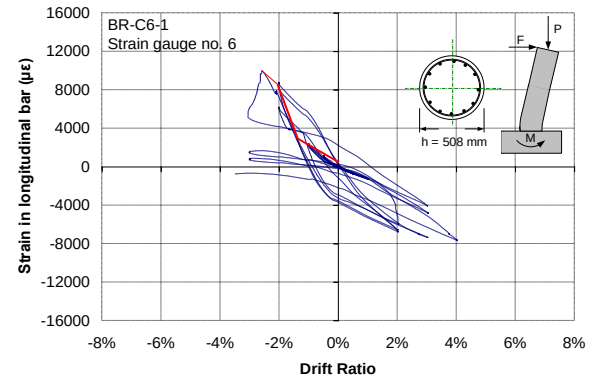


Strain gauge no. 2

**Fig. 3.20** Longitudinal reinforcement steel strains for column BR-C6-1



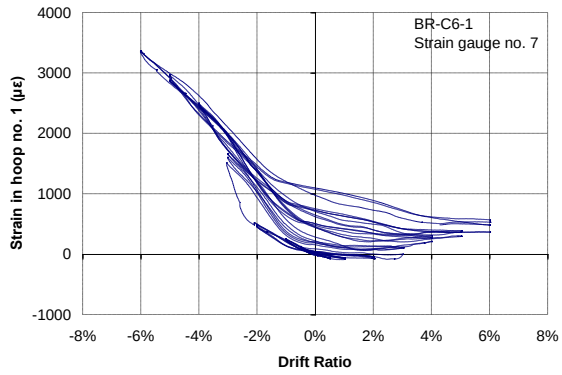
Strain gauge no. 2



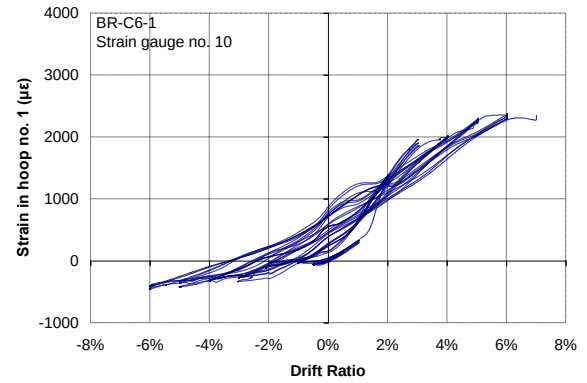
gauge no. 6

Strain

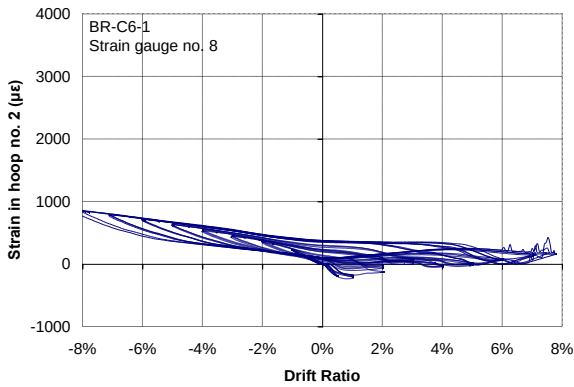
**Fig. 3.20(Cont'd)** Longitudinal reinforcement steel strains for column BR-C6-1



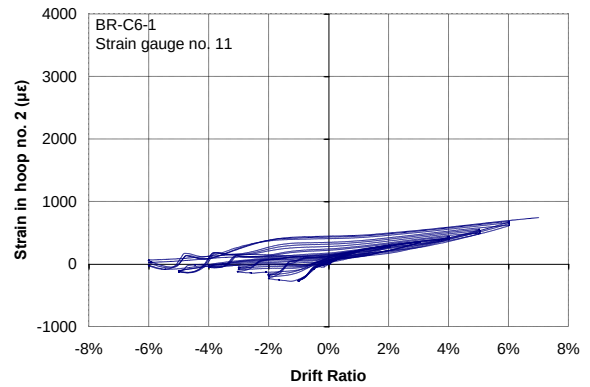
Strain gauge no. 7



Strain gauge no. 10

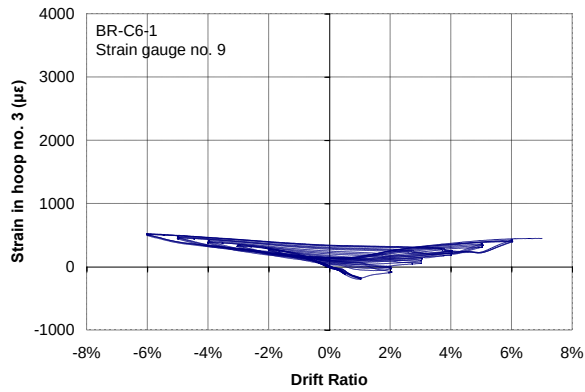


Strain gauge no. 8

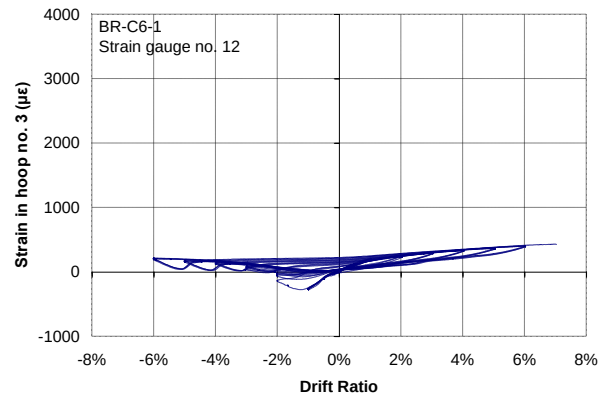


Strain gauge no. 11

**Fig. 3.21** Transverse reinforcement steel strains for column BR-C6-1

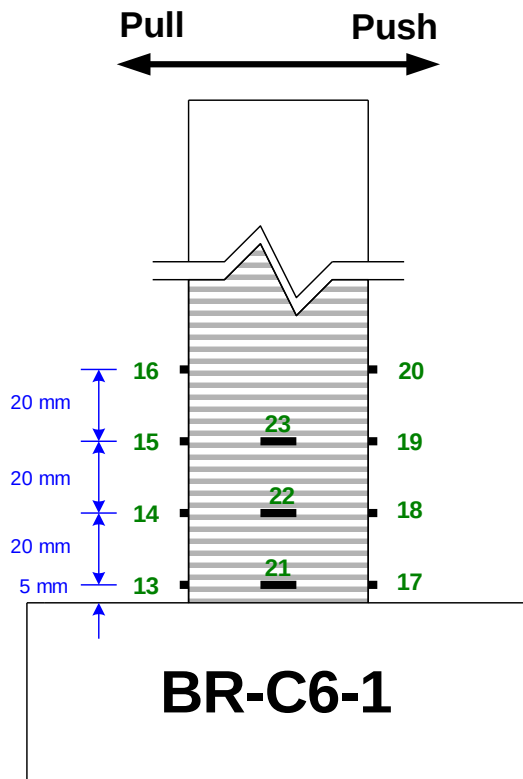


Strain gauge no. 9

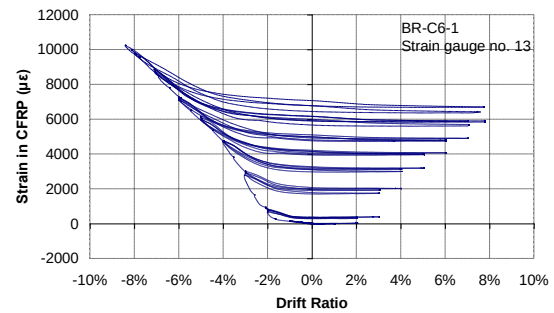


Strain gauge no. 12

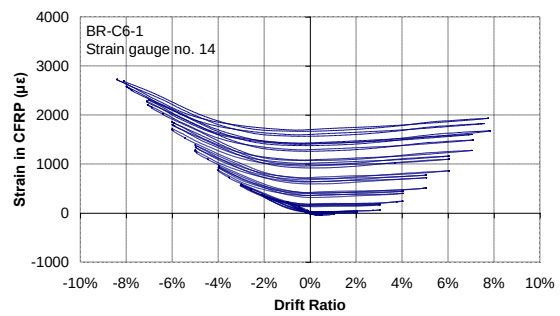
**Fig. 3.21(Cont'd)** Transverse reinforcement steel strains for column BR-C6-1



Strain gauge location on CFRP sheets

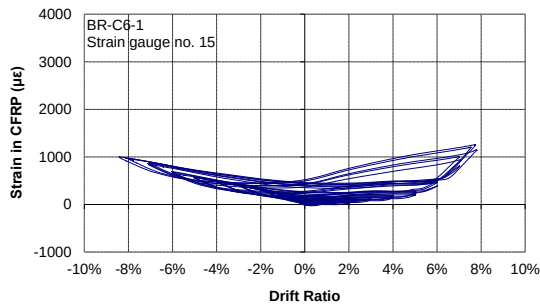


Strain gauge no. 1

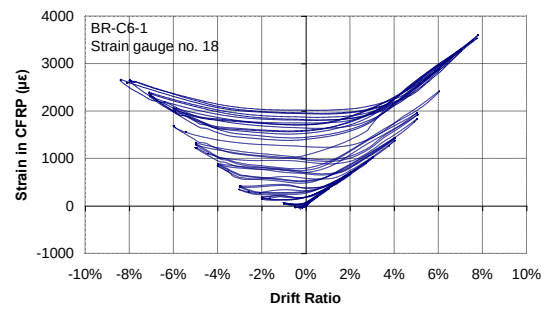


Strain gauge no. 14

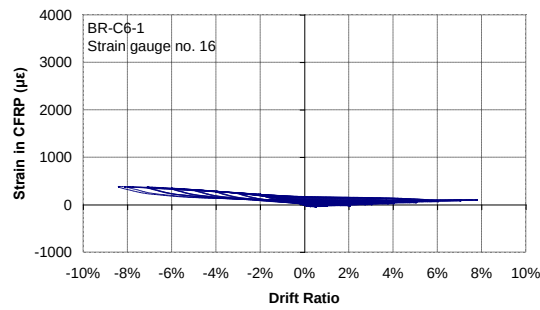
**Fig. 3.22** CFRP sheet strains for column BR-C6-1



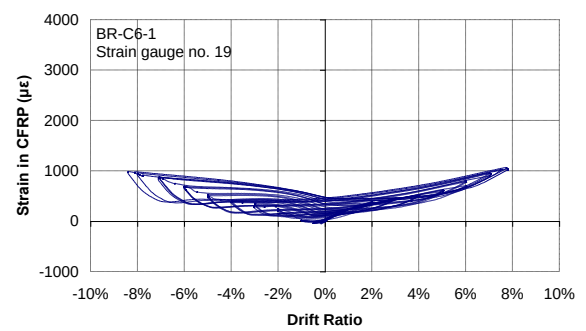
Strain gauge no. 15



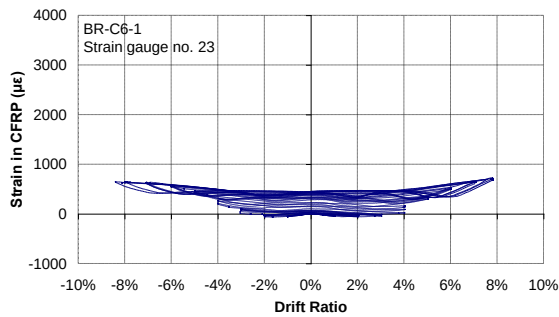
Strain gauge no. 18



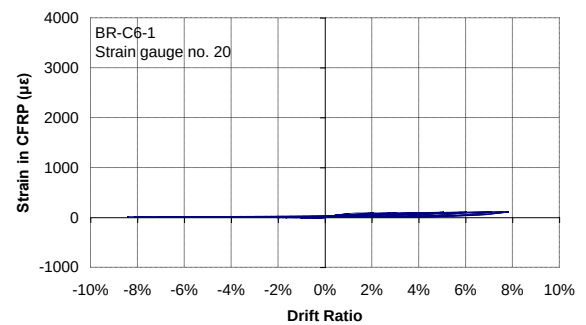
Strain gauge no. 16



Strain gauge no. 19

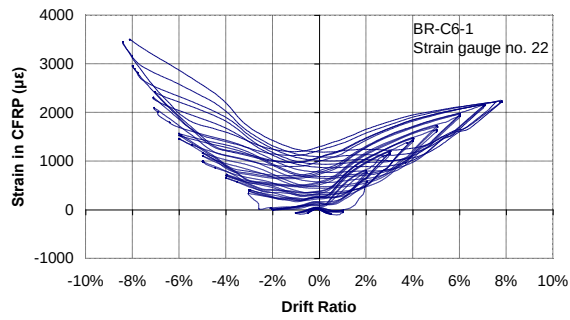


Strain gauge no. 23

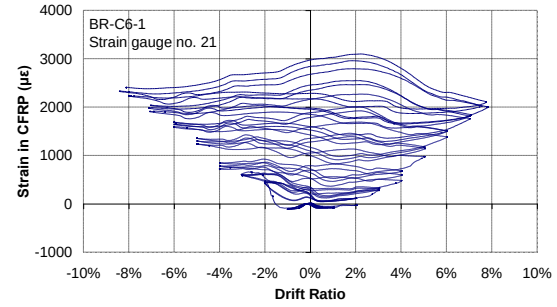


Strain gauge no. 20

**Fig. 3.22(Cont'd)** CFRP sheet strains for column BR-C6-1

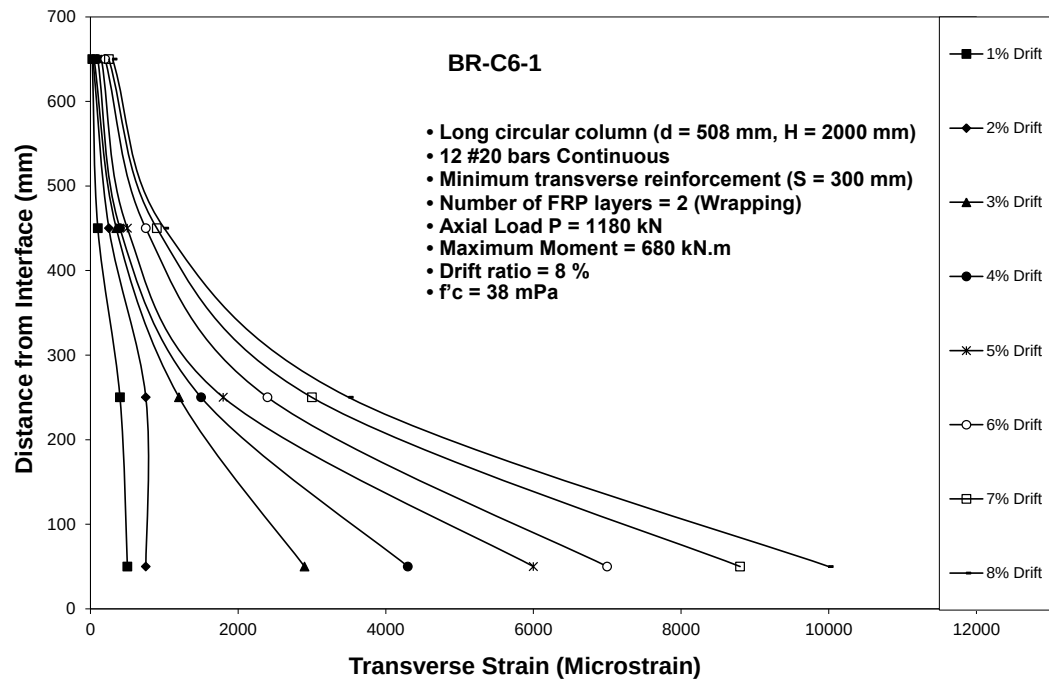


Strain gauge no. 22



Strain gauge no.21

**Fig. 3.22(Cont'd)** CFRP sheet strains for column BR-C6-1



**Fig. 3.23** Fiber strain distribution along column BR-C6-1





**(a) 4% Drift**



**(b) 5% Drift**



**(c) 6% Drift**

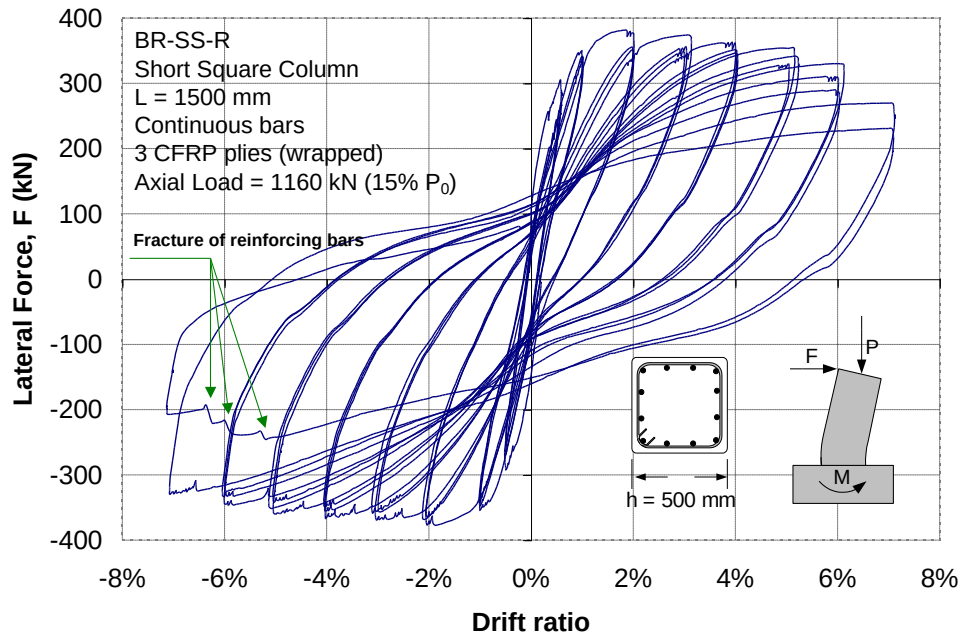
**Fig. 3.24** Behavior of column BR-SS-R at different drift levels



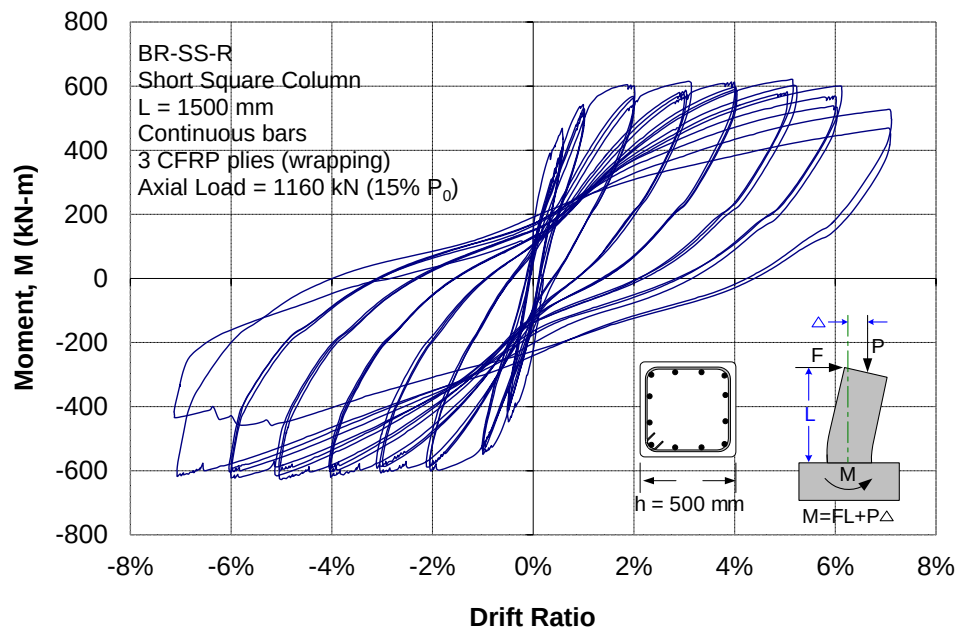


(d) At the end of test

**Fig. 3.24(Cont'd)** Behavior of column BR-SS-R at different drift levels

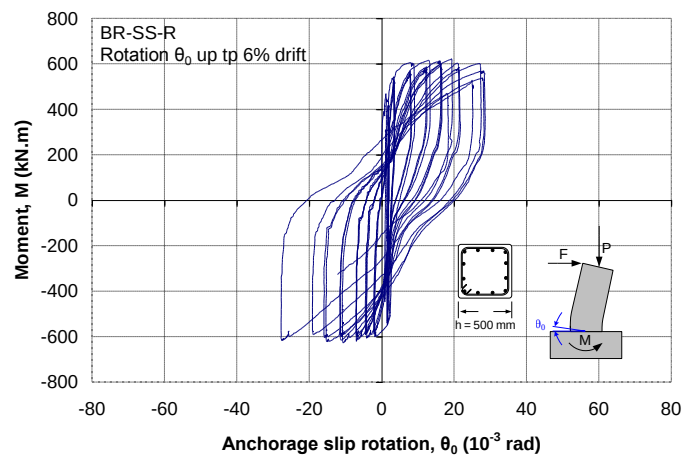
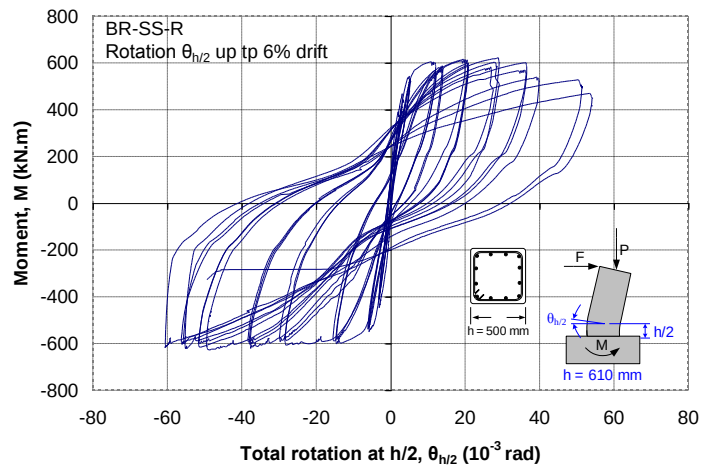
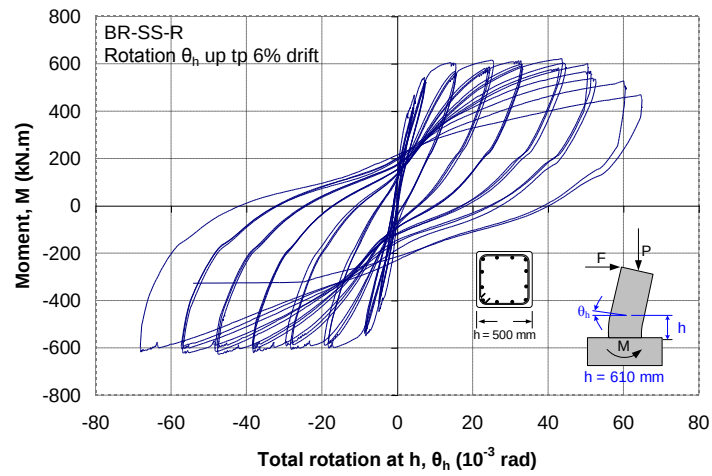


a) Force-Drift relationship

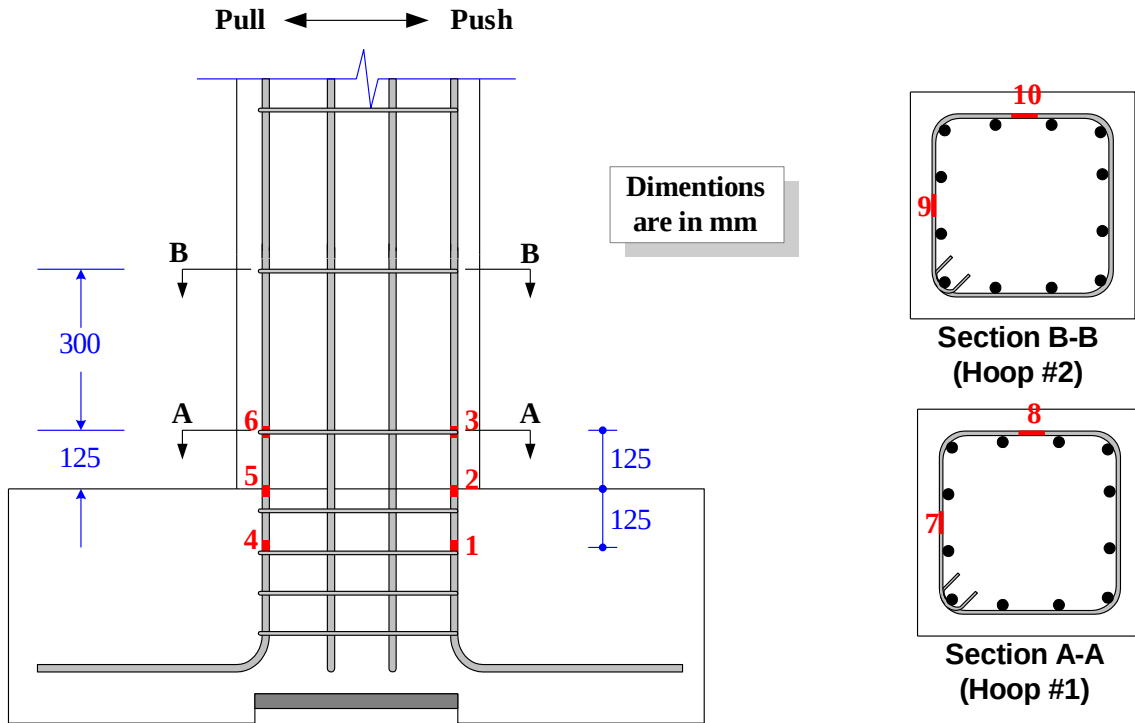


b) Moment-Drift relationship

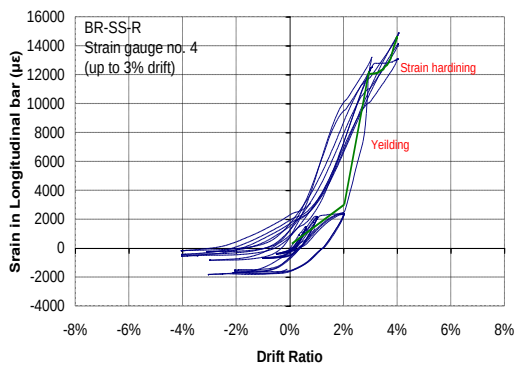
**Fig. 3.25** Hysteretic relationship for column BR-SS-R



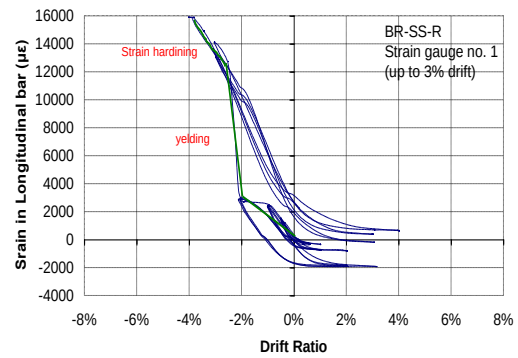
**Fig. 3.26** Rotation of the hinging region for column BR-SS-R



Strain gauges location on reinforcement steel

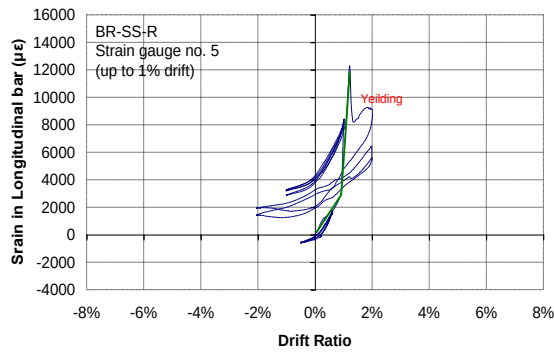


Strain gauge no. 4

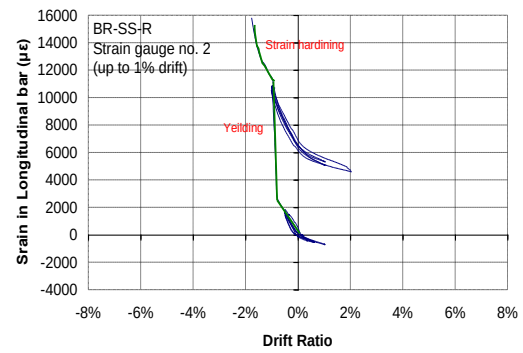


Strain gauge no. 1

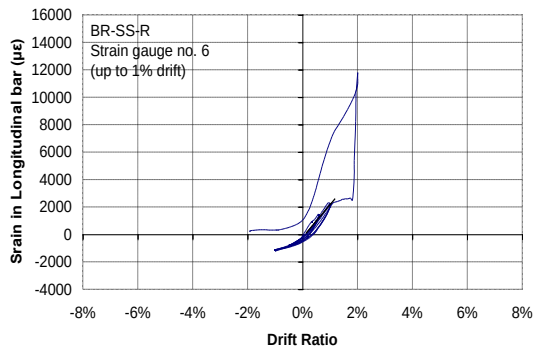
**Fig. 3.27** Longitudinal reinforcement steel strains for column BR-SS-R



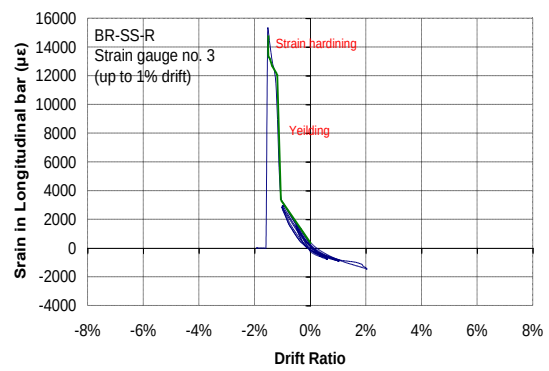
Strain gauge no. 5



Strain gauge no. 2

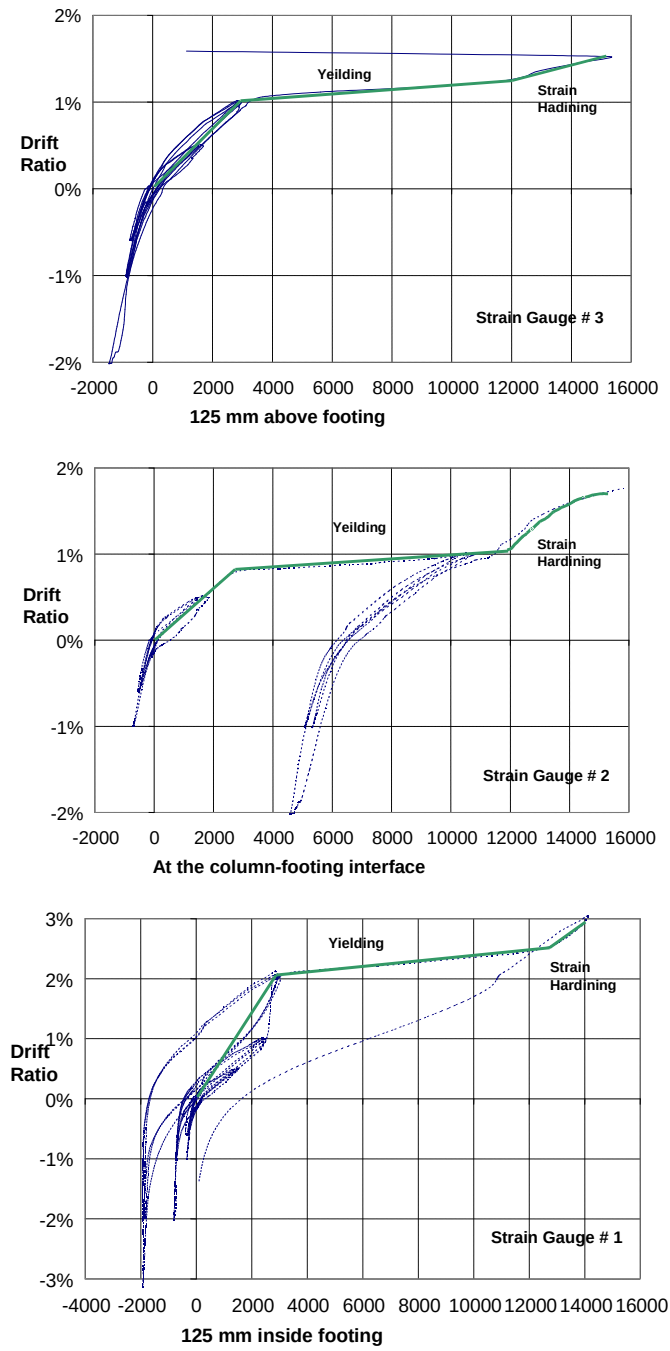


Strain gauge no. 6

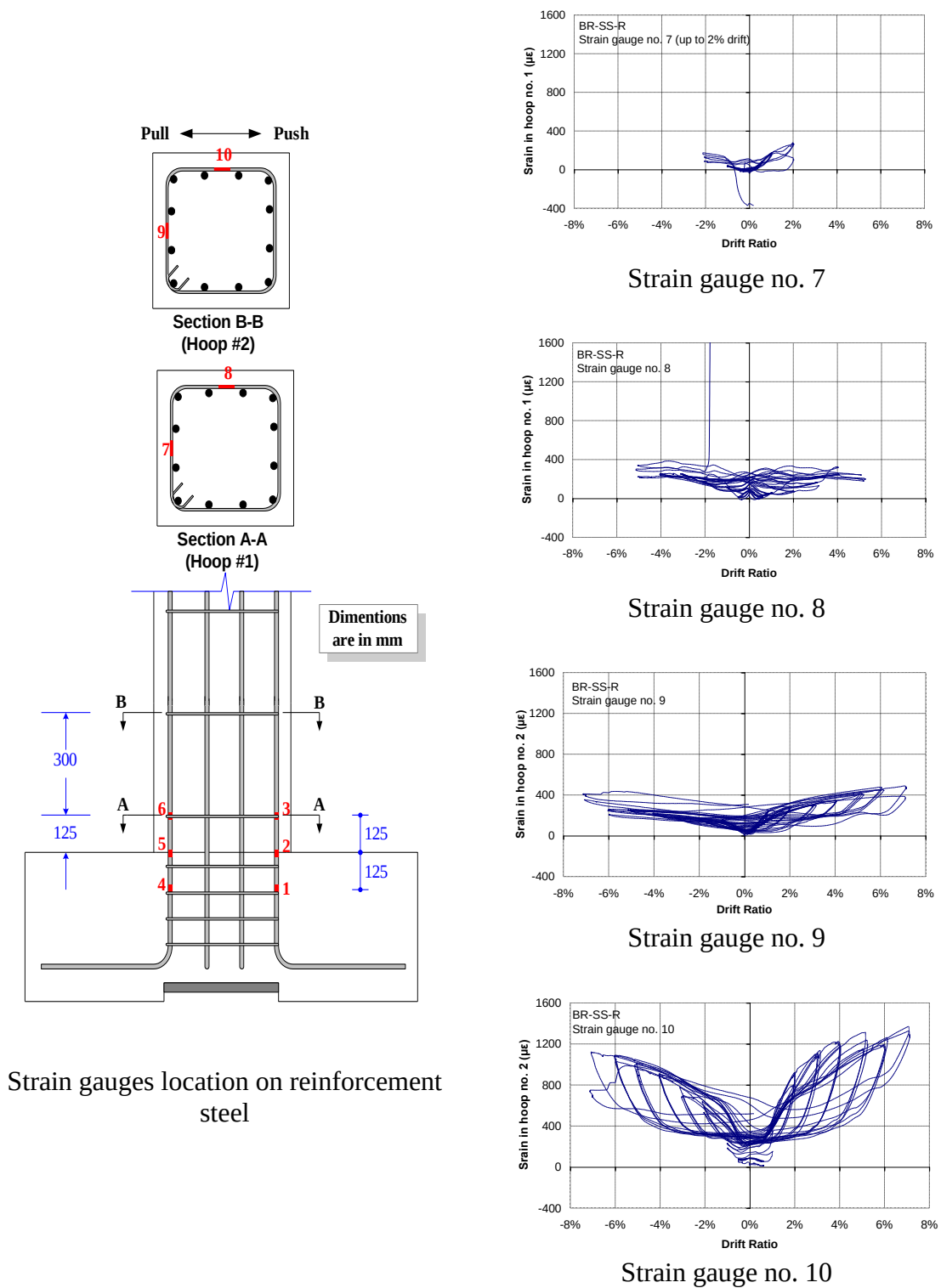


Strain gauge no. 3

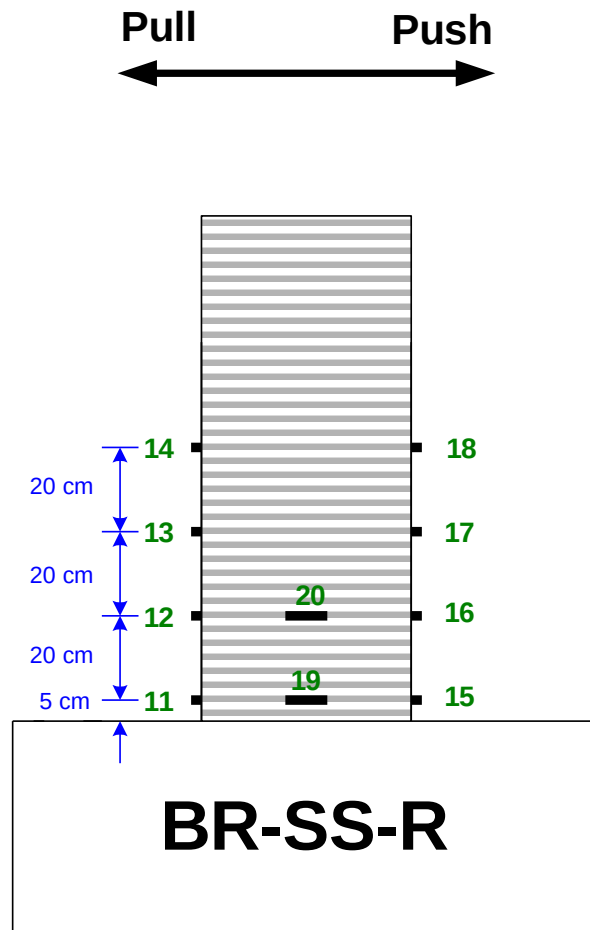
**Fig. 3.27(Cont'd)** Longitudinal reinforcement steel strains for column BR-SS-R



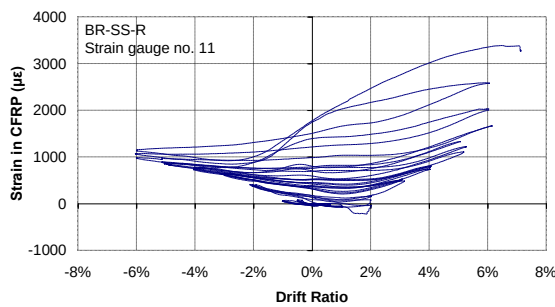
**Fig. 3.28** Yield penetration in longitudinal reinforcement steel along plastic hinge region for column BR-SS-R



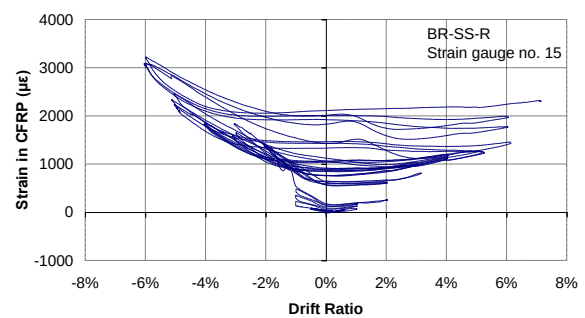
**Fig. 3.29** Transverse reinforcement steel strains for column BR-SS-R



Strain gauges location on CFRP sheet



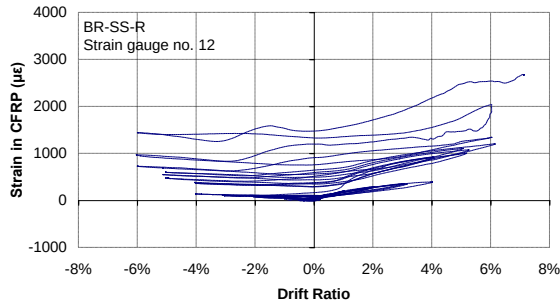
Strain gauge no. 11



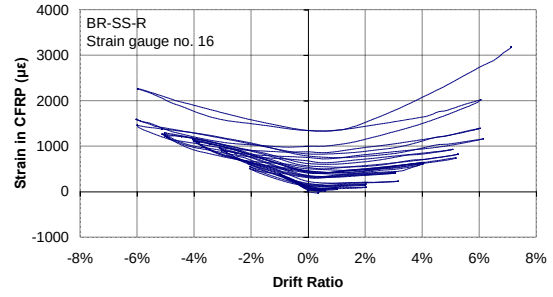
Strain gauge no. 15

**Fig. 3.30** CFRP sheet strains for column BR-SS-R

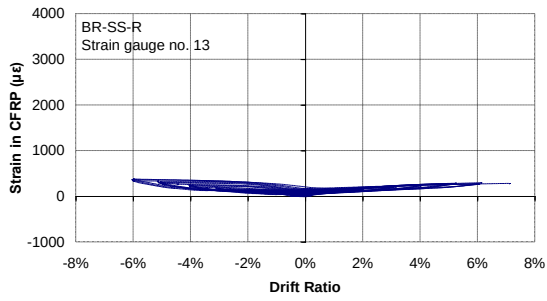




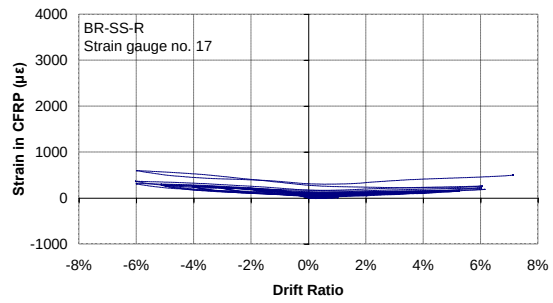
Strain gauge no. 12



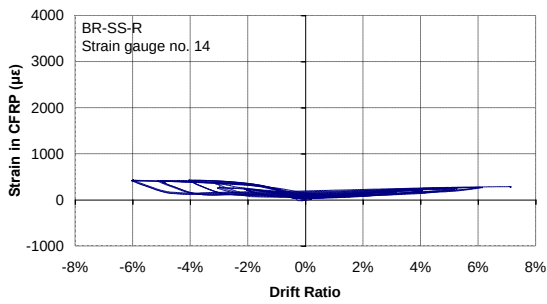
Strain gauge no. 16



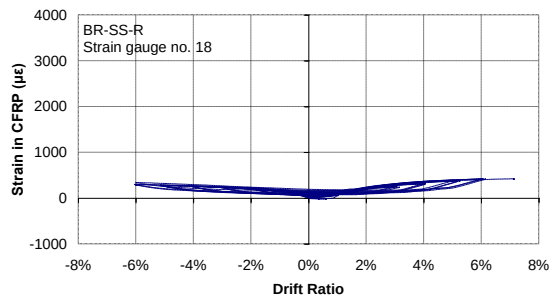
Strain gauge no. 13



Strain gauge no. 17

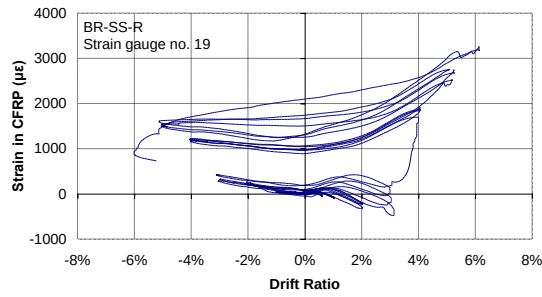


Strain gauge no. 14

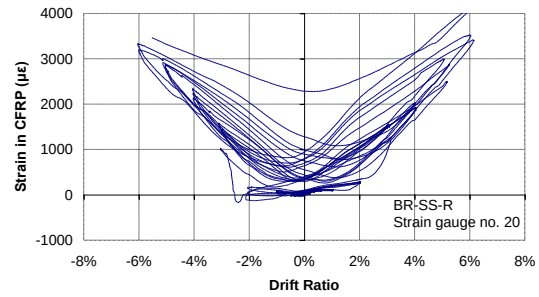


Strain gauge no. 18

**Fig. 3.30(Cont'd)** CFRP sheet strains for column BR-SS-R

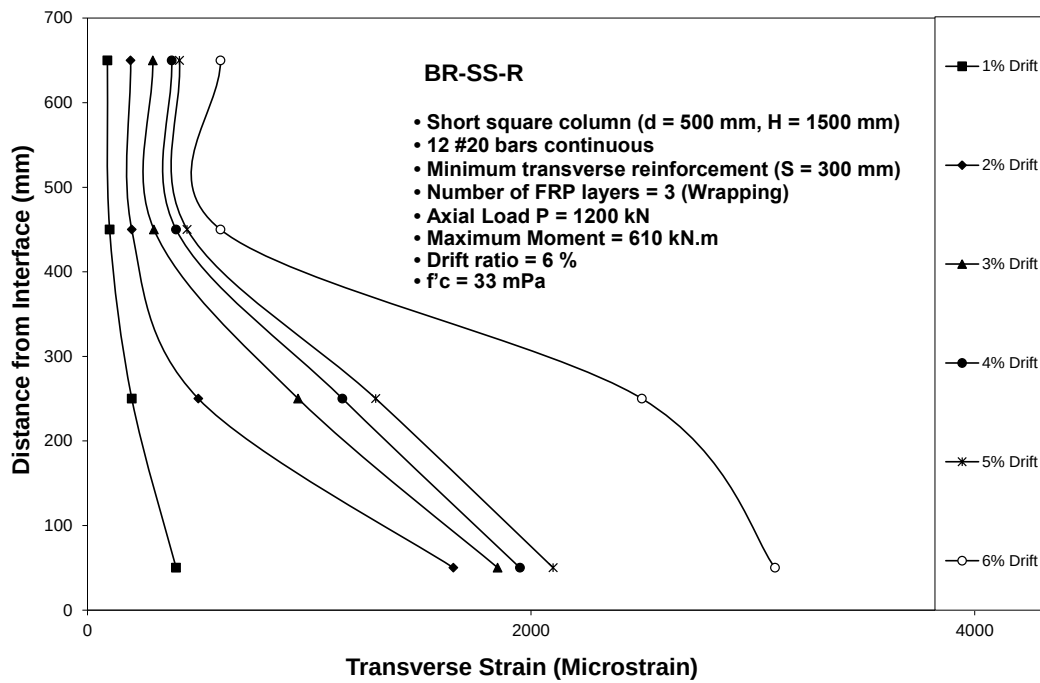


Strain gauge no. 19



Strain gauge no. 20

**Fig. 3.30(Cont'd)** CFRP sheet strains for column BR-SS-R



**Fig. 3.31** Fiber Strain distribution along column BR-SS-R



**(a)** 3% Drift



**(b)** 4% Drift



**(c)** 5% Drift



**(d)** 6% Drift

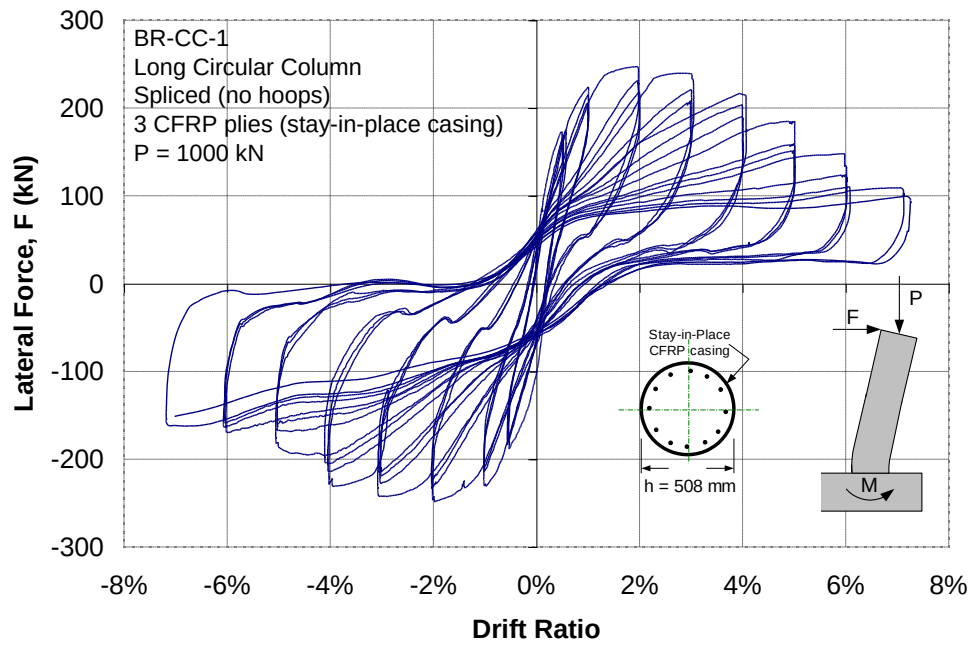


**(e)** 6% Drift

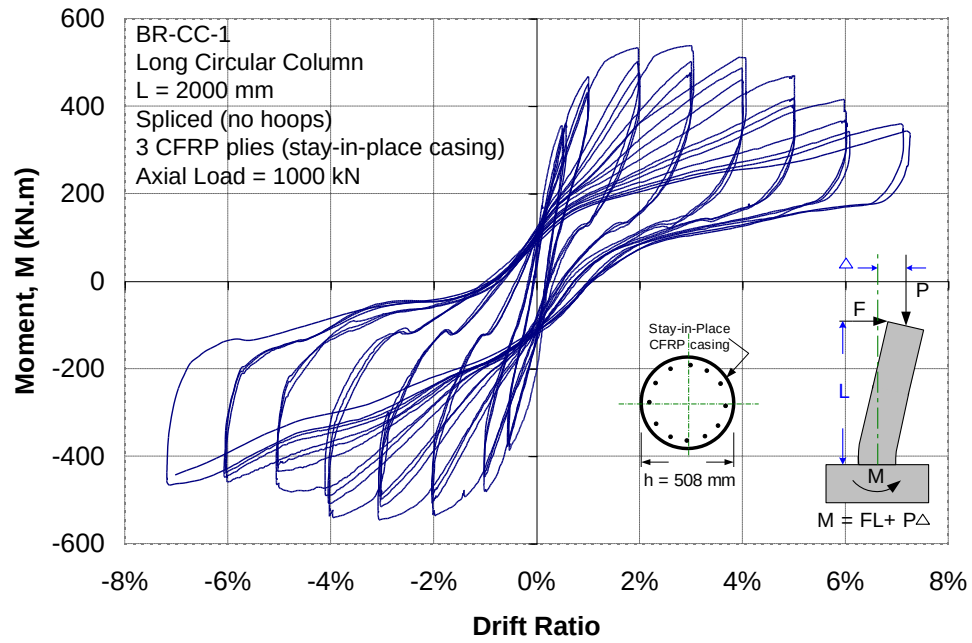


**(f)** 7% Drift

**Fig. 3.32** Behavior of column BR-CC-1 at different drift levels

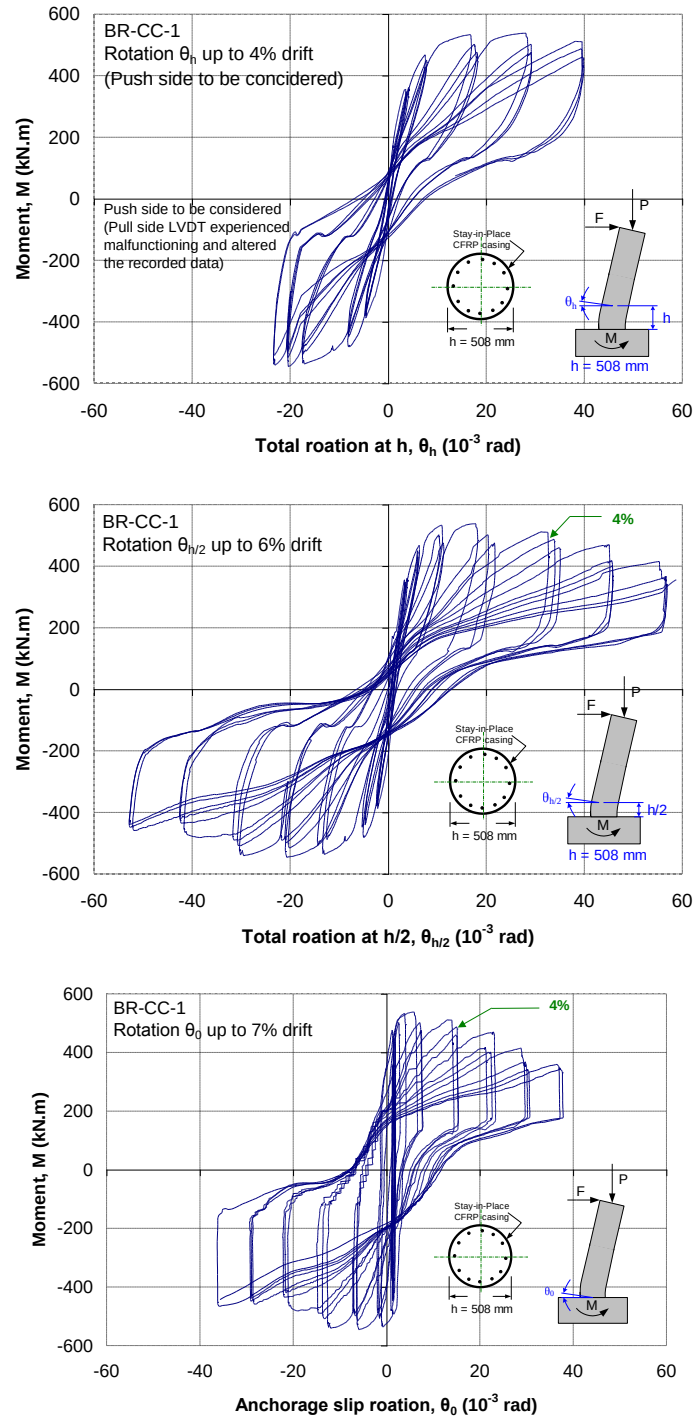


a) Force-Drift relationship

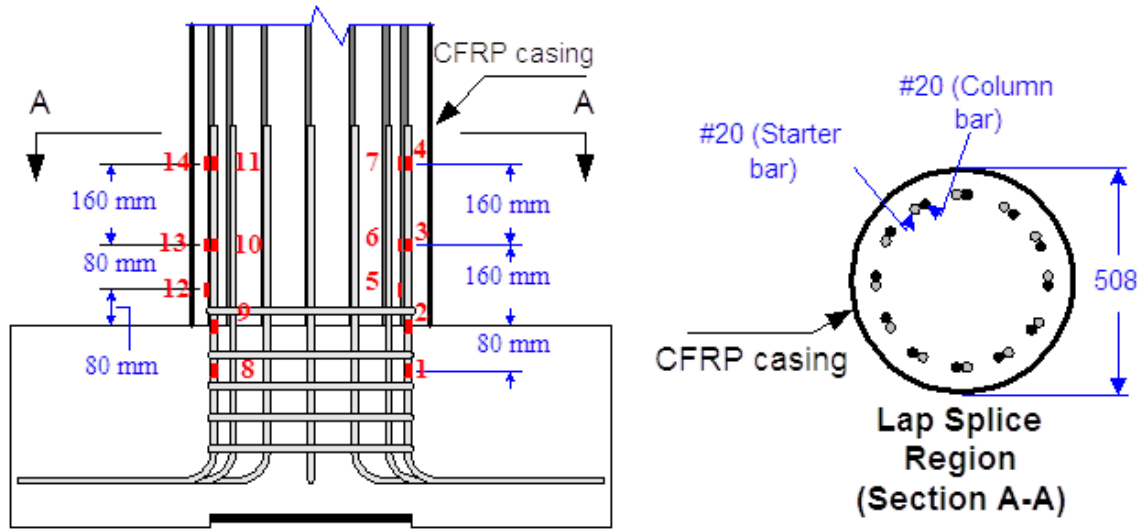


b) Moment-Drift relationship

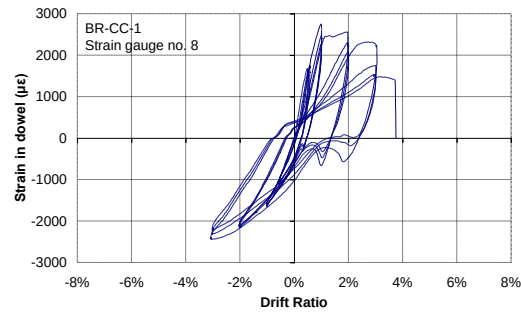
**Fig. 3.33** Hysteretic relationship for column BR-CC-1



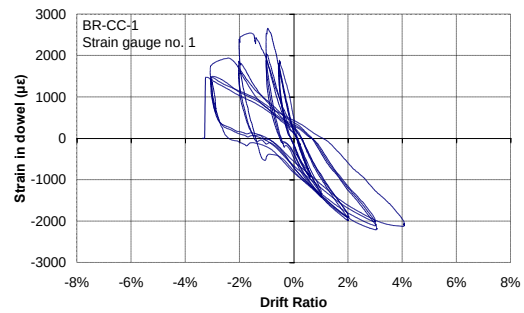
**Fig. 3.34** Rotation of the hinging region for column BR-CC-1



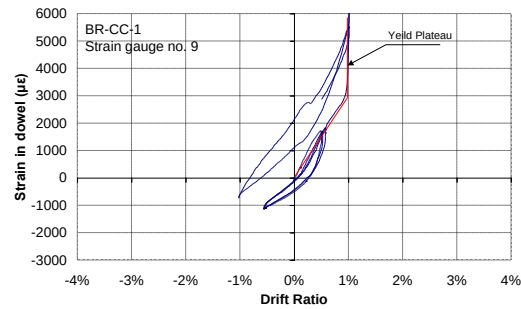
Strain gauge location



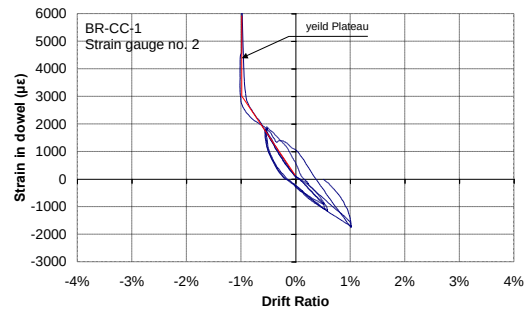
Strain gauge no. 8



Strain gauge no.1

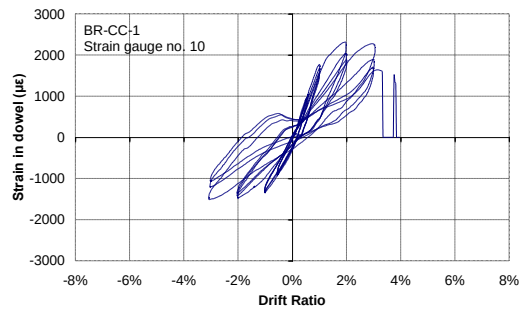


Strain gauge no.9

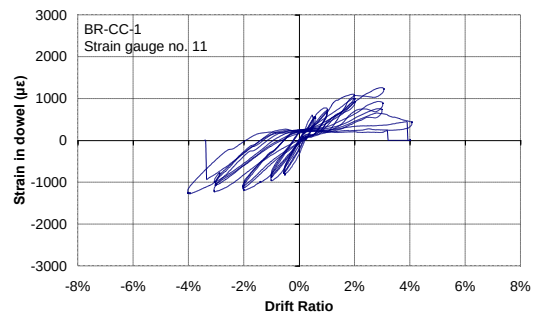


Strain gauge no. 2

Fig. 3.35 Dowel strains for column BR-CC-1

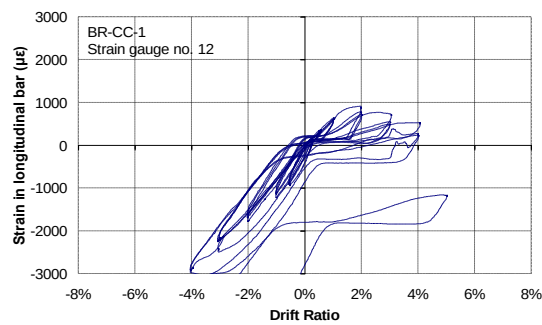


Strain gauge no.10

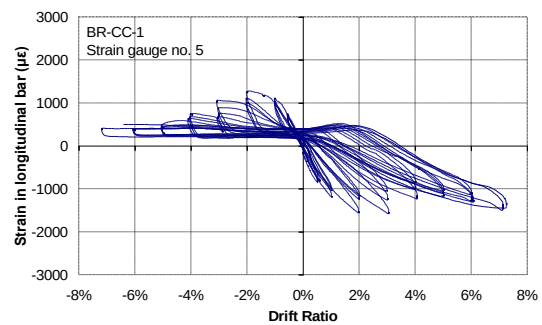


Strain gauge no. 11

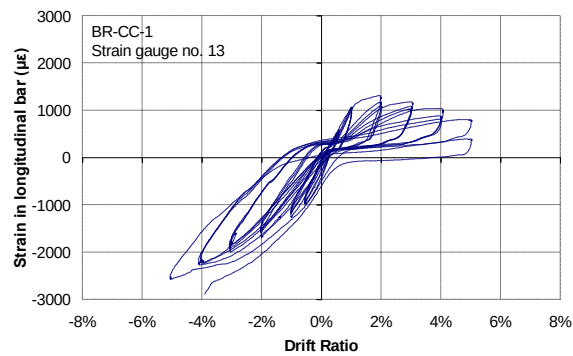
**Fig. 3.35(Cont'd)** Dowel strains for column BR-CC-1



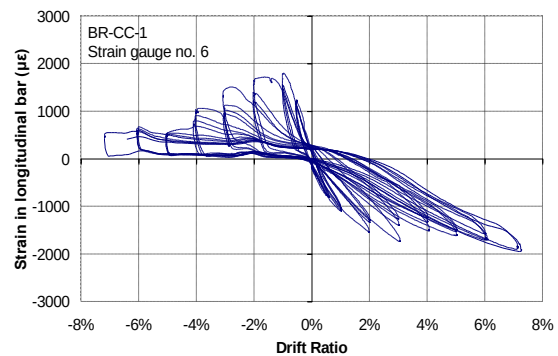
Strain gauge no.12



Strain gauge no.5



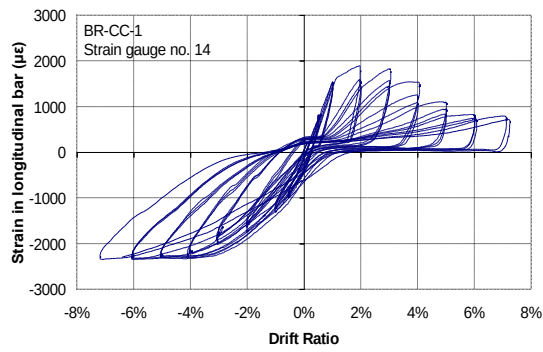
Strain gauge no.13



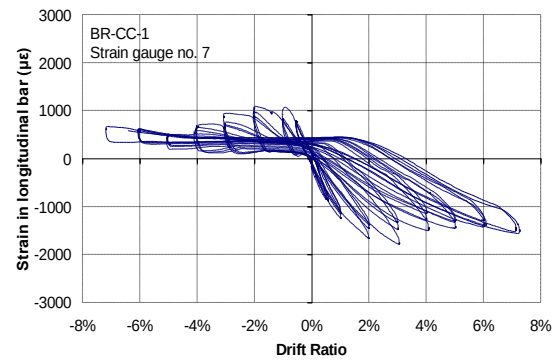
Strain gauge no.6

**Fig. 3.36** Longitudinal reinforcement strains for column BR-CC-1



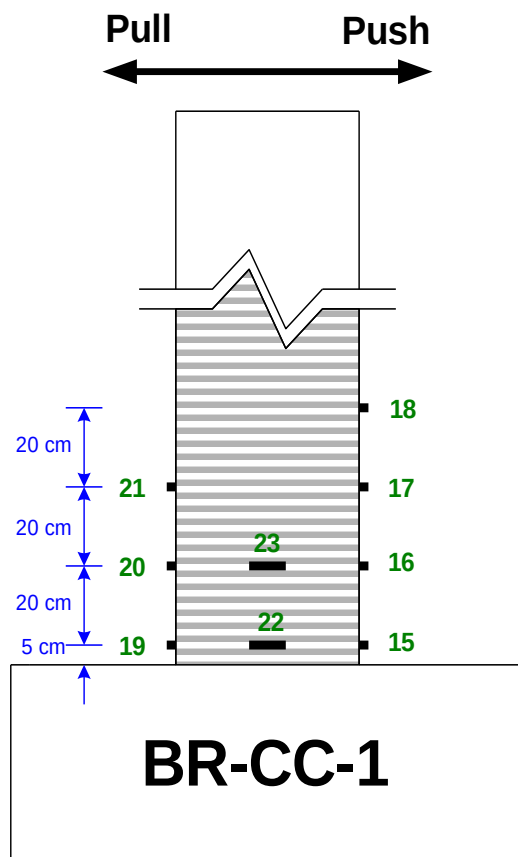


Strain gauge no.14

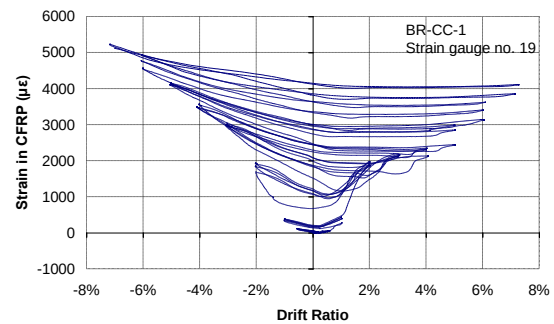


Strain gauge no.7

**Fig. 3.36 (Cont'd)** Longitudinal reinforcement strains for column BR-CC-1



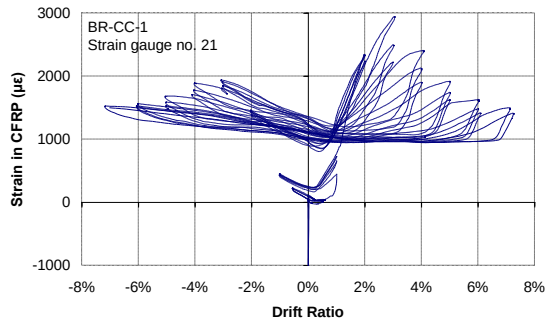
Strain gauge location on CFRP sheet



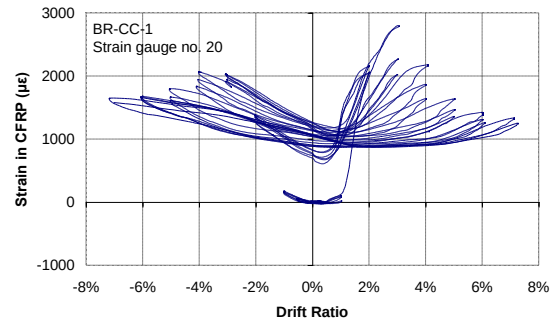
Strain gauge no19

**Fig. 3.37** CFRP sheet strains for column BR-CC-1

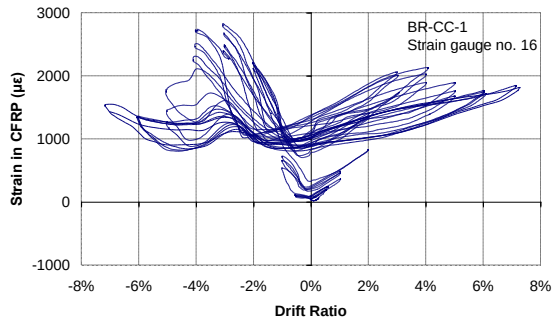




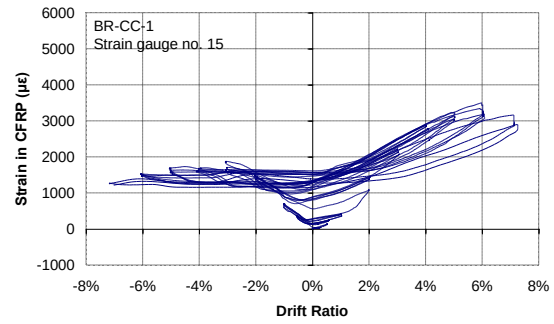
Strain gauge no21



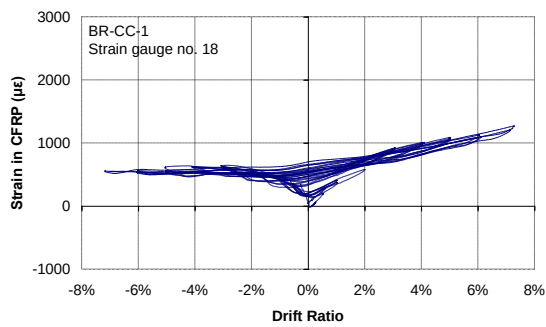
Strain gauge no.20



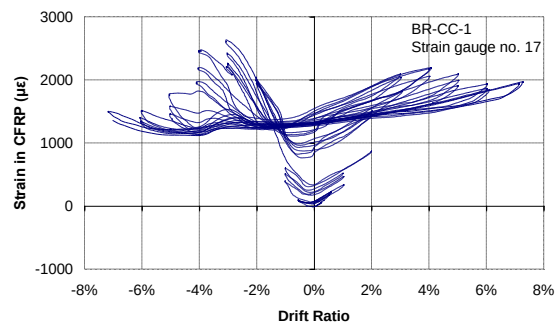
Strain gauge no.16



Strain gauge no.15

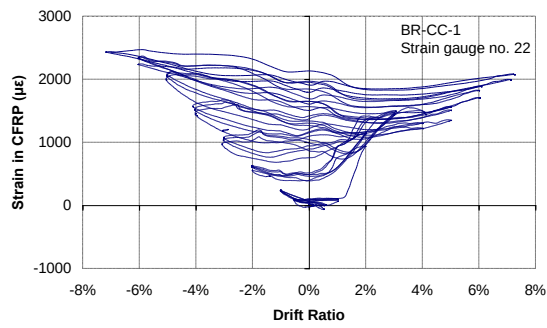


Strain gauge no18

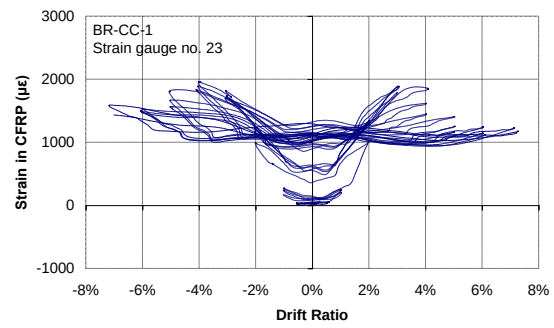


Strain gauge no.17

**Fig. 3.37 (Cont'd)** CFRP sheet strains for column BR-CC-1

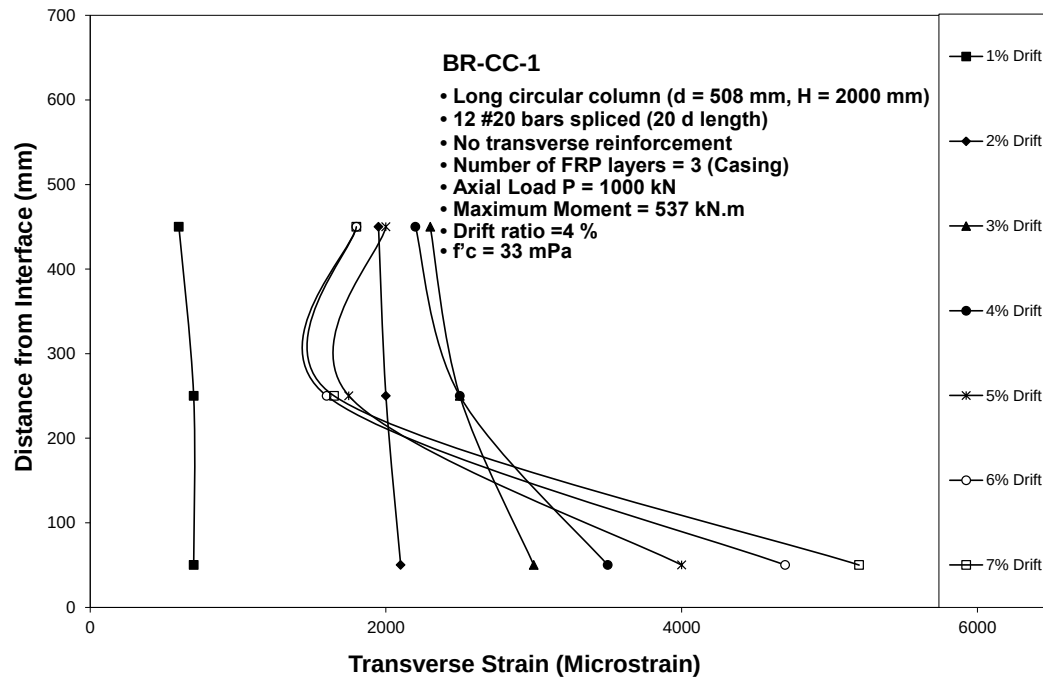


Strain gauge no.22



Strain gauge no.23

**Fig. 3.37(Cont'd)** CFRP sheet strains for column BR-CC-1



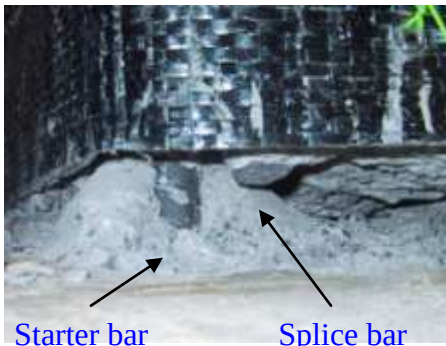
**Fig. 3.38** Fiber Strain distribution along column BR-CC-1



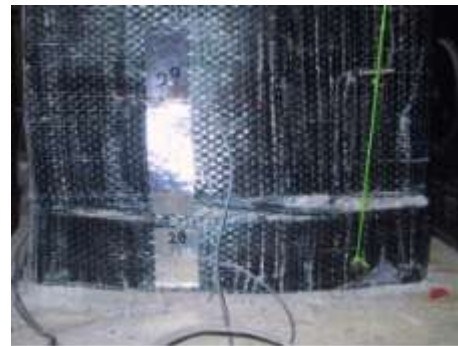
(a)



(b)



(c)



(d)

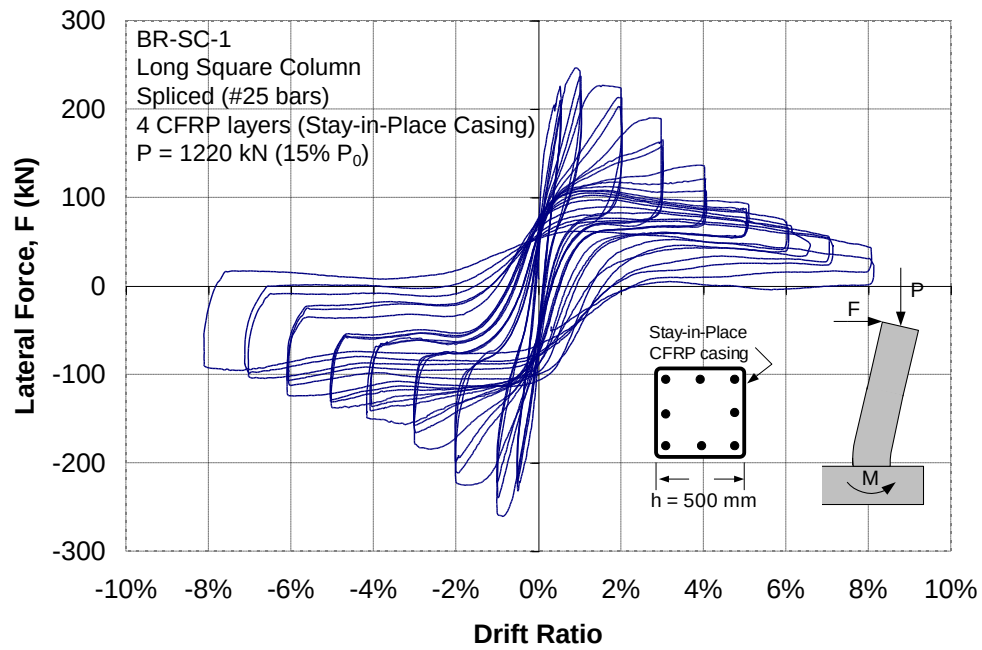


(e)

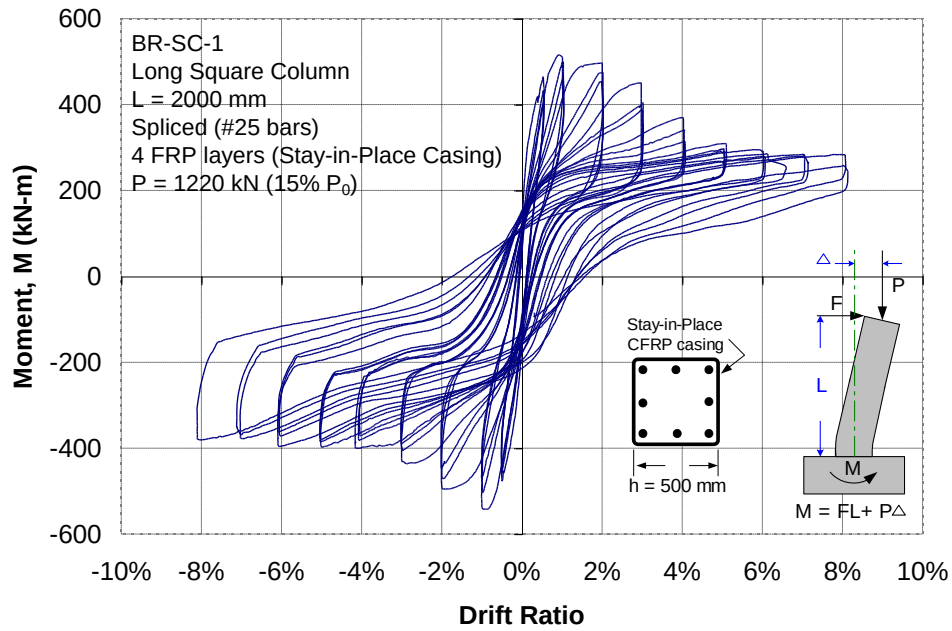


(f)

**Fig. 3.39** Behavior of column BR-SC-1 at different drift levels

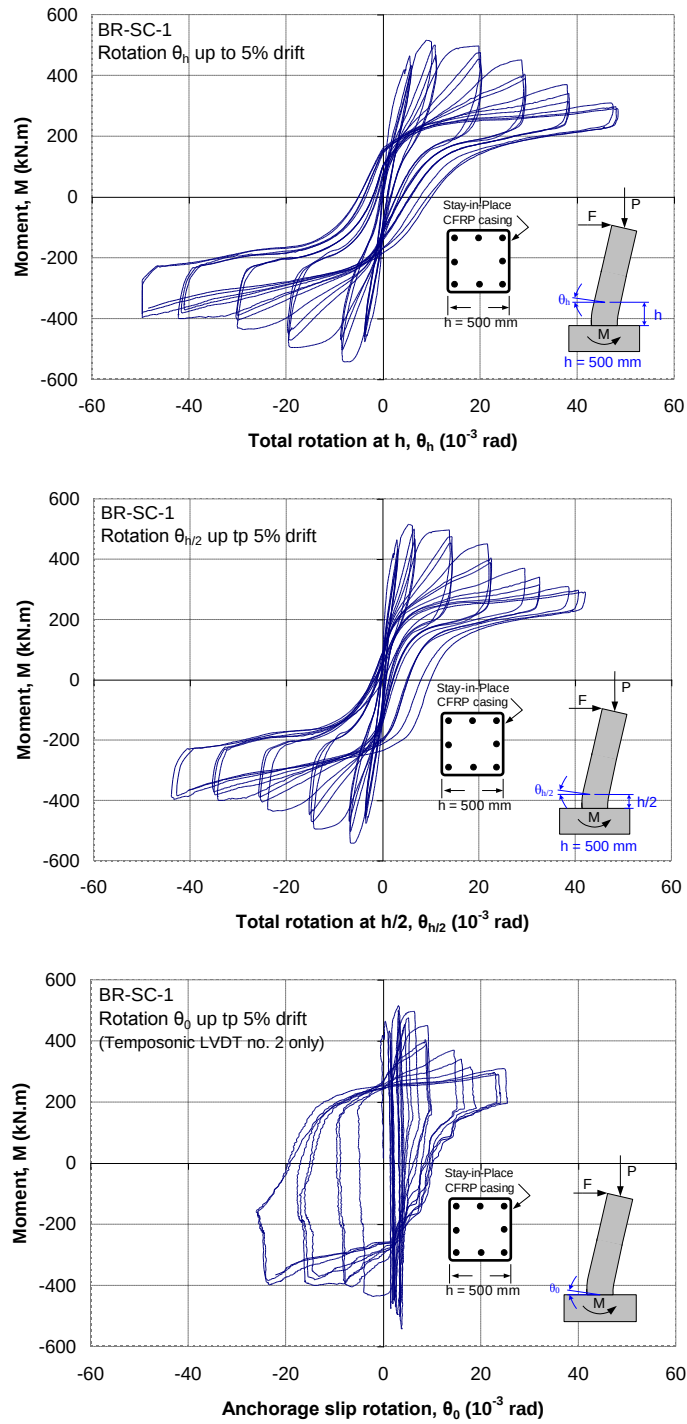


a) Force-Drift relationship

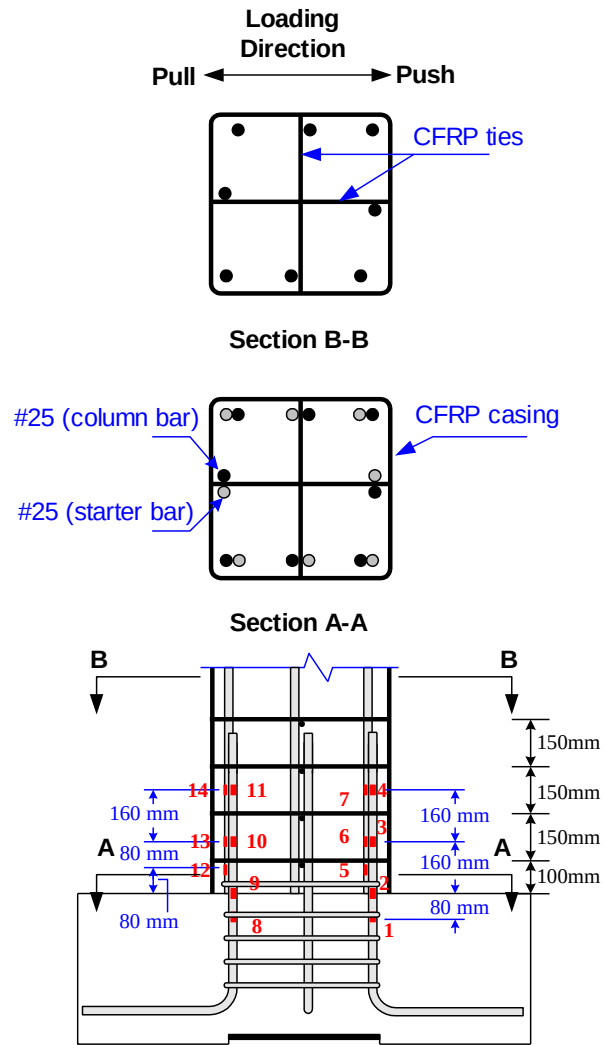


b) Moment-Drift relationship

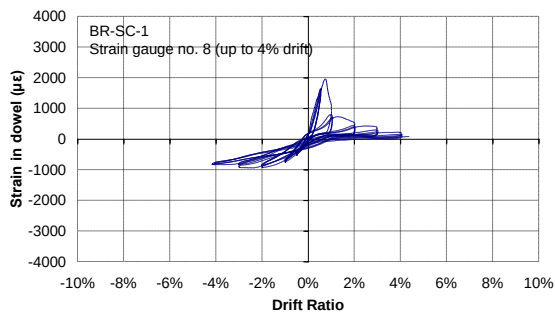
**Fig. 3.40** Hysteretic relationship for column BR-SC-1



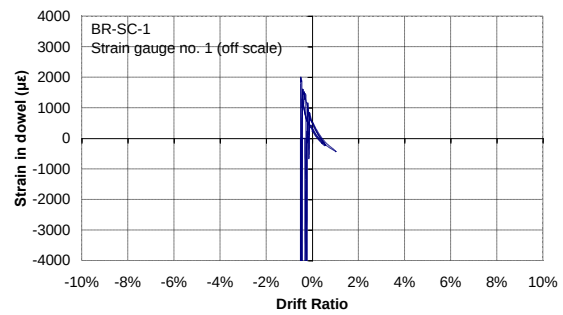
**Fig. 3.41** Rotation of the hinging region for column BR-SC-1



Strain gauge locations on dowel bars in column BR-SC-1

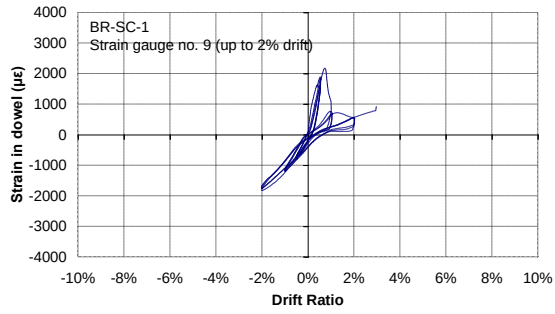


Strain gauge no. 8

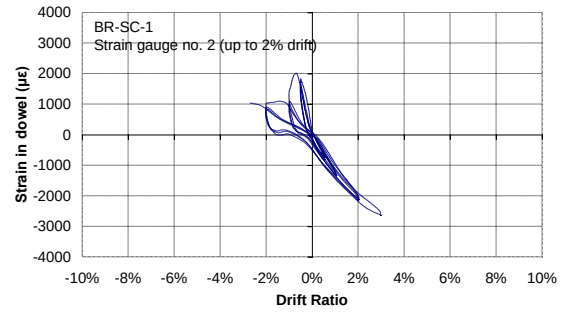


Strain gauge no. 1

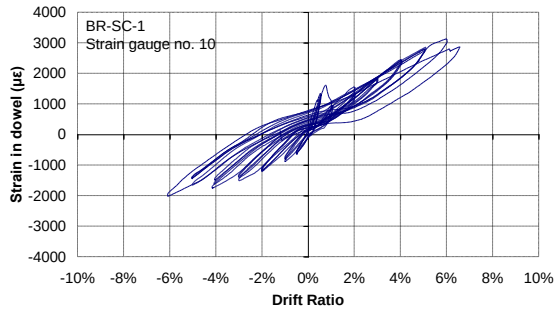
**Fig. 3.42** Dowel strains for column BR-SC-1



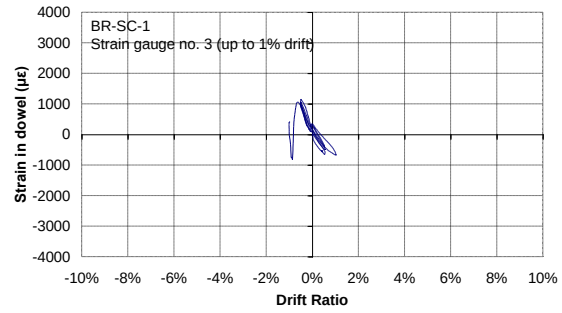
Strain gauge no. 9



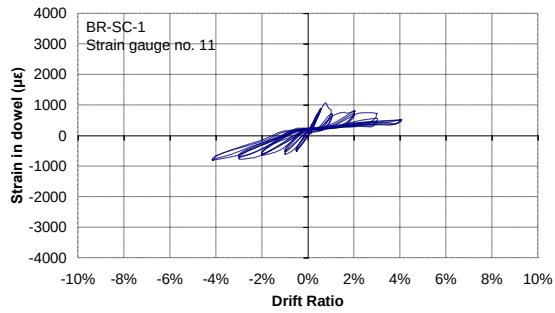
Strain gauge no. 2



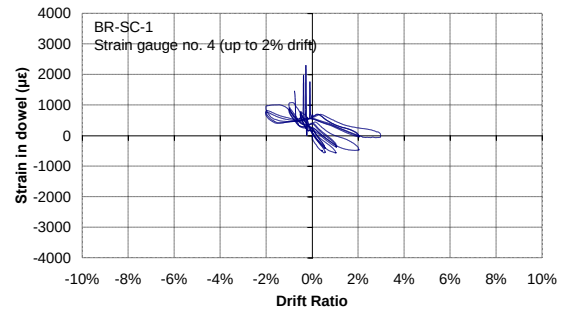
Strain gauge no. 10



Strain gauge no. 3

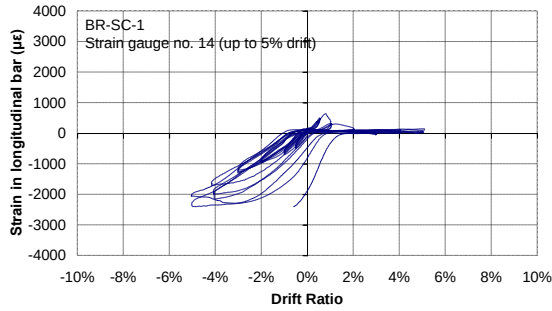


Strain gauge no. 11

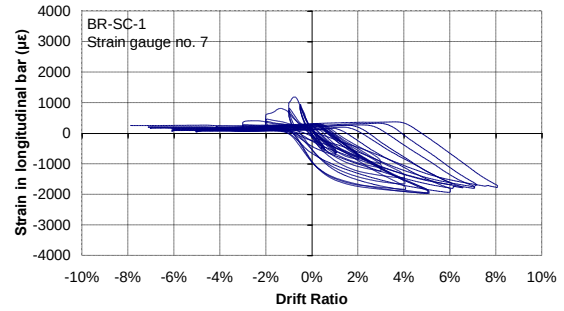


Strain gauge no. 4

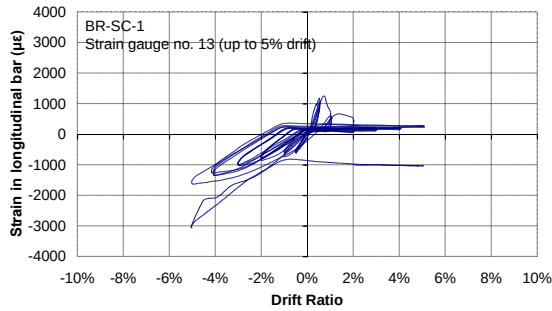
**Fig. 3.42(Cont'd)** Dowel strains for column BR-SC-1



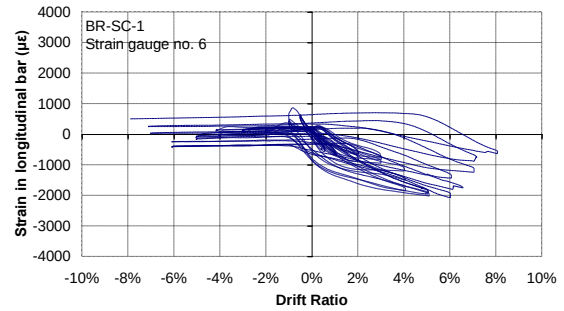
Strain gauge no. 14



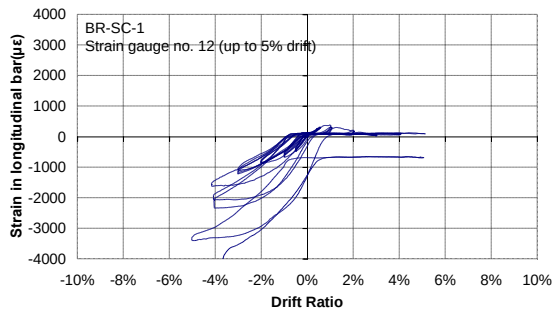
Strain gauge no. 7



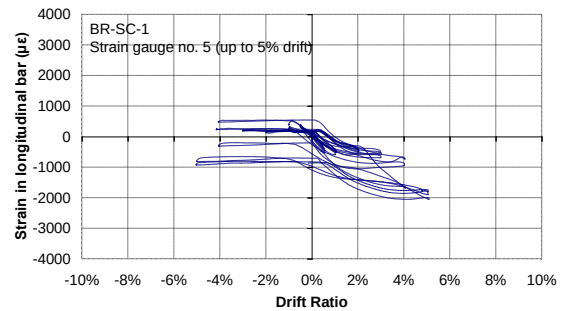
Strain gauge no. 13



Strain gauge no. 6



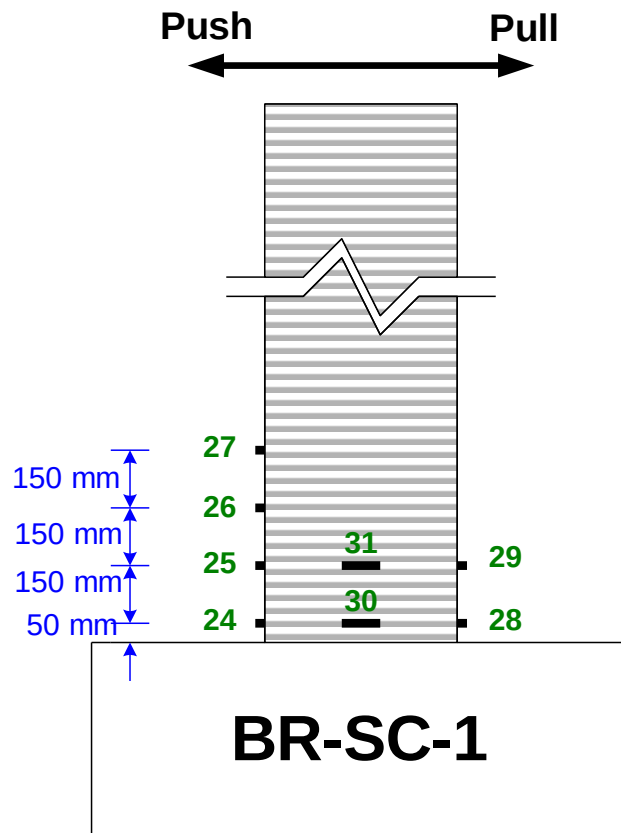
Strain gauge no. 12



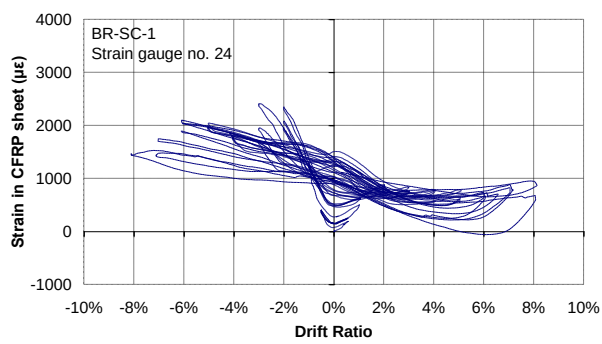
Strain gauge no. 5

**Fig. 3.43** Longitudinal reinforcement steel strains for column BR-SC-1

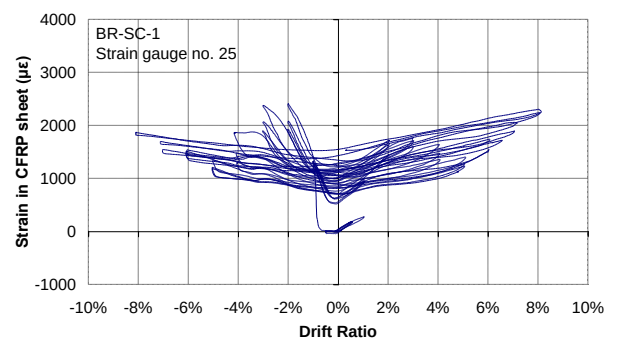




Strain gauge location on CFRP sheets

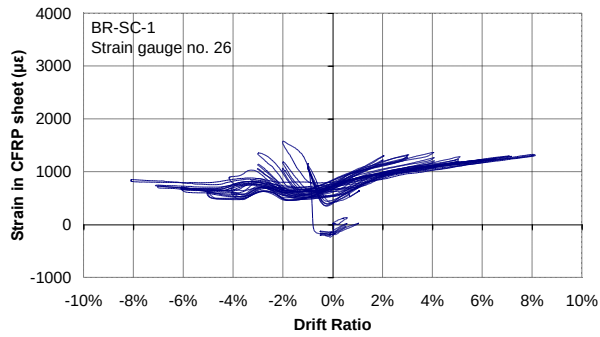


Strain gauge no. 24

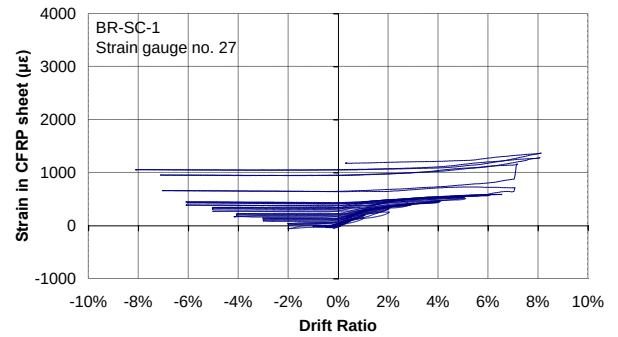


Strain gauge no. 25

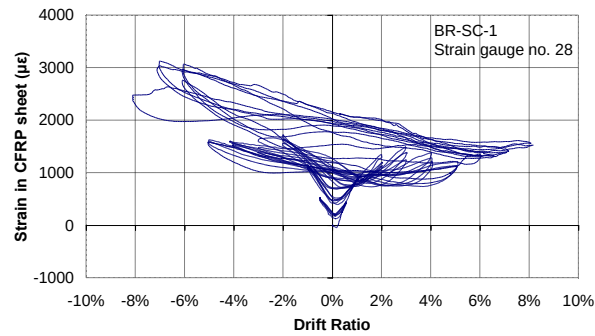
**Fig. 3.44** CFRP sheet strains for column BR-SC-1



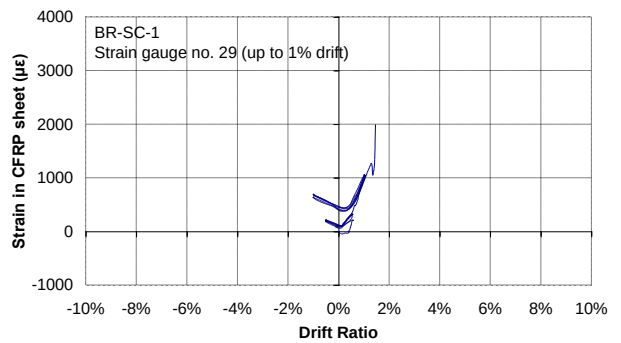
Strain gauge no. 26



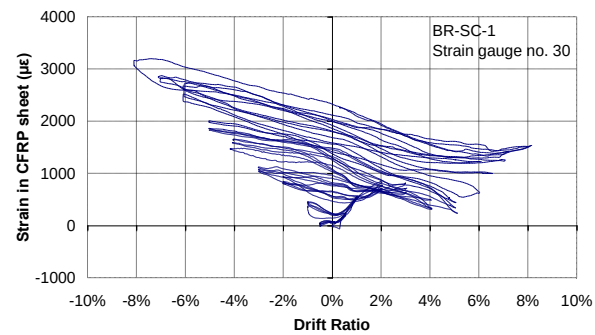
Strain gauge no. 27



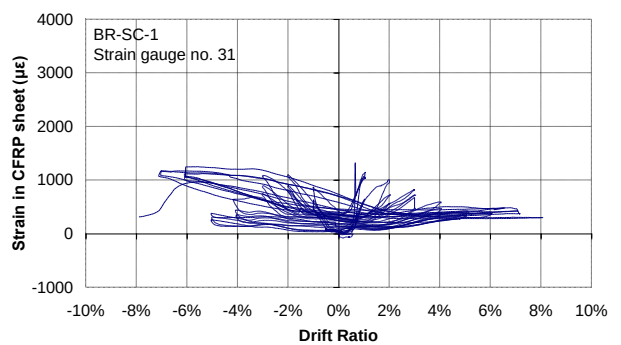
Strain gauge no. 28



Strain gauge no. 24

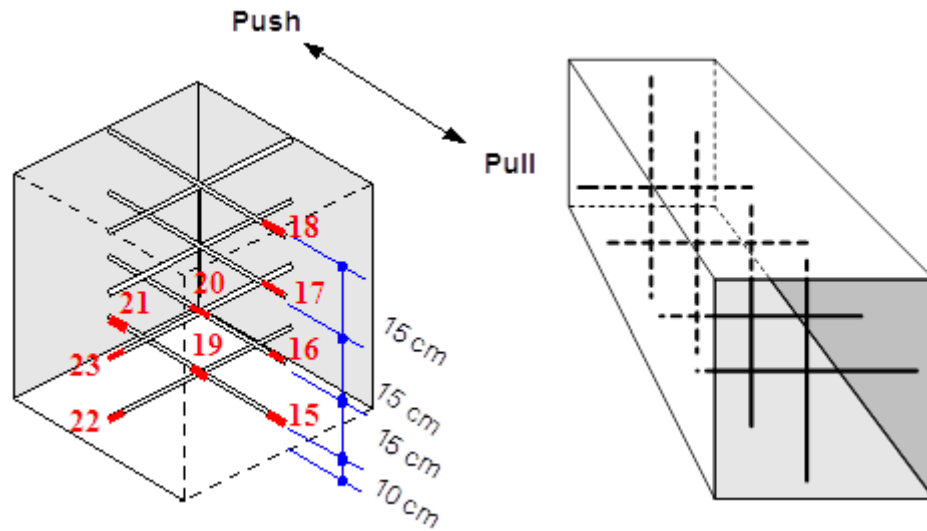


Strain gauge no. 30

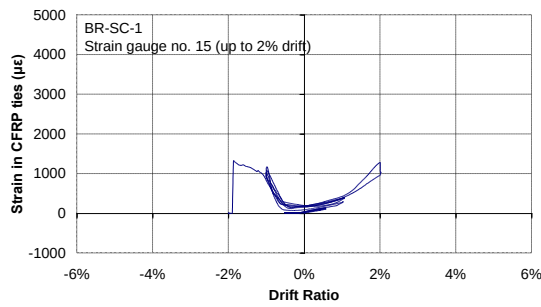


Strain gauge no. 31

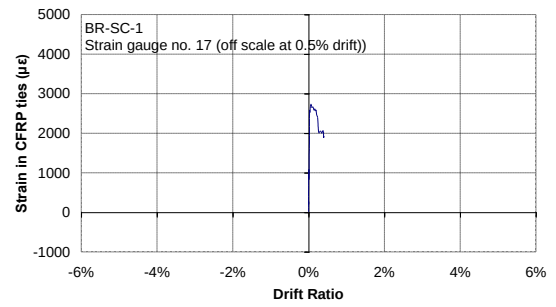
**Fig. 3.44(Cont'd)** CFRP sheet strains for column BR-SC-1



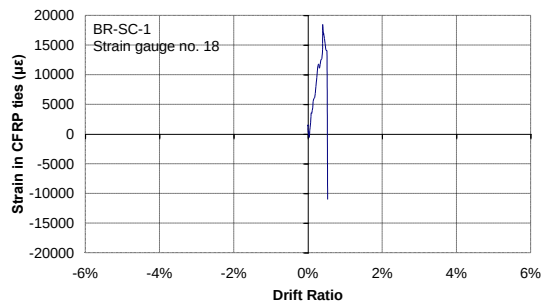
Strain gauges location on CFRP crossties



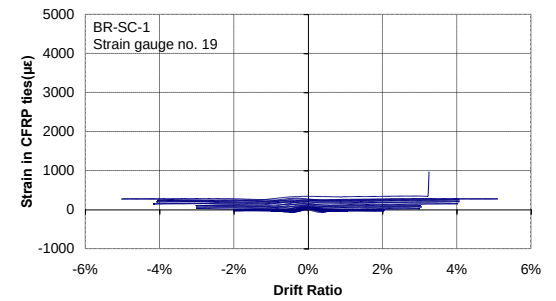
Strain gauge no. 15



Strain gauge no. 17

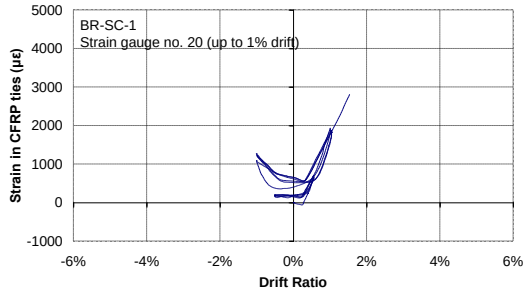


Strain gauge no. 18

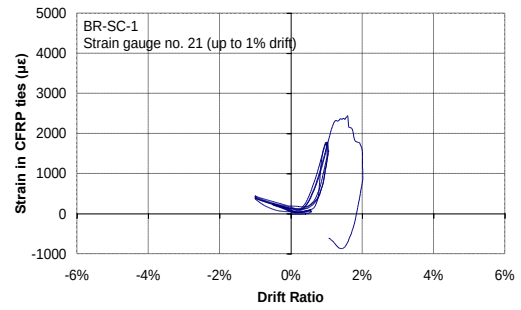


Strain gauge no. 19

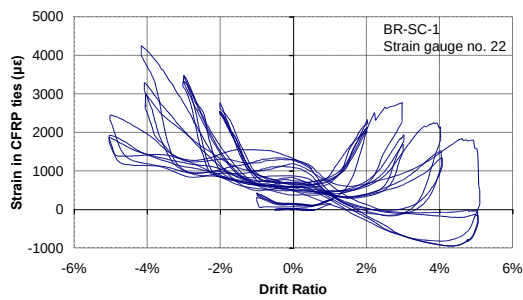
**Fig. 3.45** Strain in CFRP crossties for column BR-SC-1



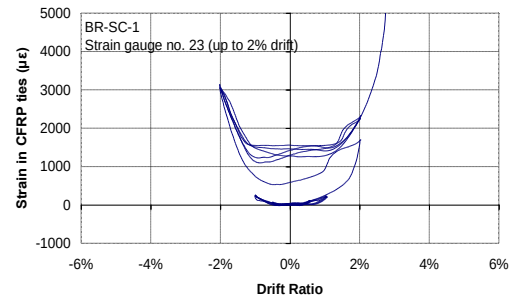
Strain gauge no. 20



Strain gauge no. 21

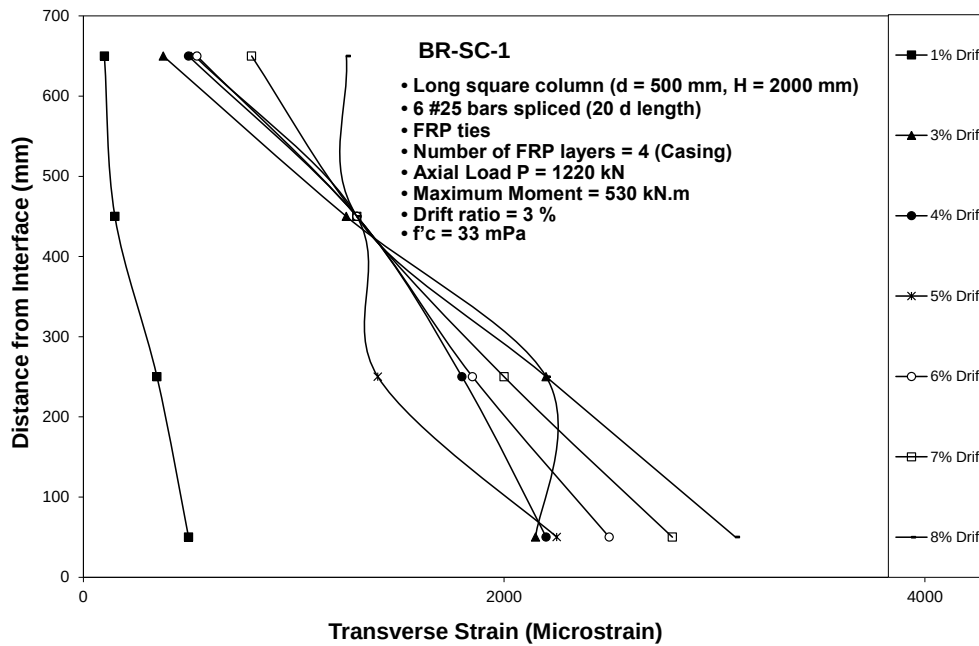


Strain gauge no. 22



Strain gauge no. 23

**Fig. 3.45(Cont'd)** Strain in CFRP crossties for column BR-SC-1



**Fig. 3.46** Fiber Strain distribution along column BR-SC-1



(a)



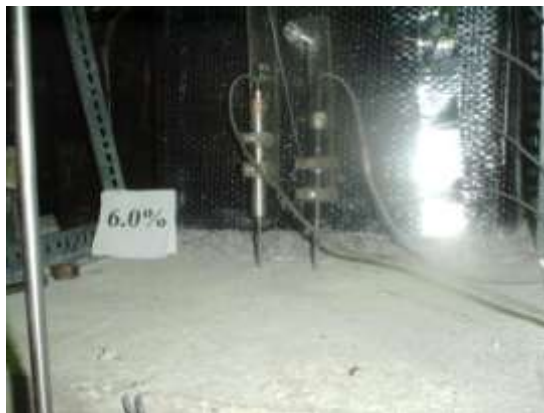
(b)



(c)



(d)



(e)

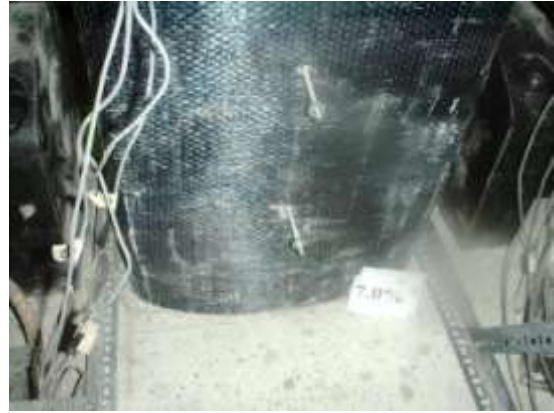


(f)

**Fig 3.47** Behavior of column BR-SC-2 at different drift levels



(g)



(h)



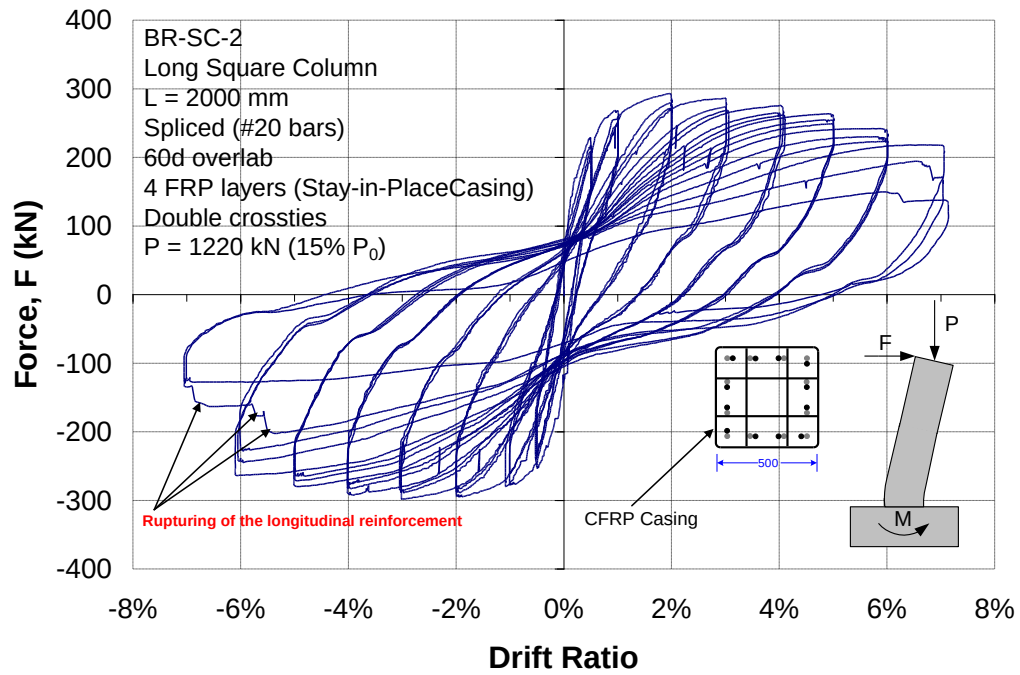
(i)



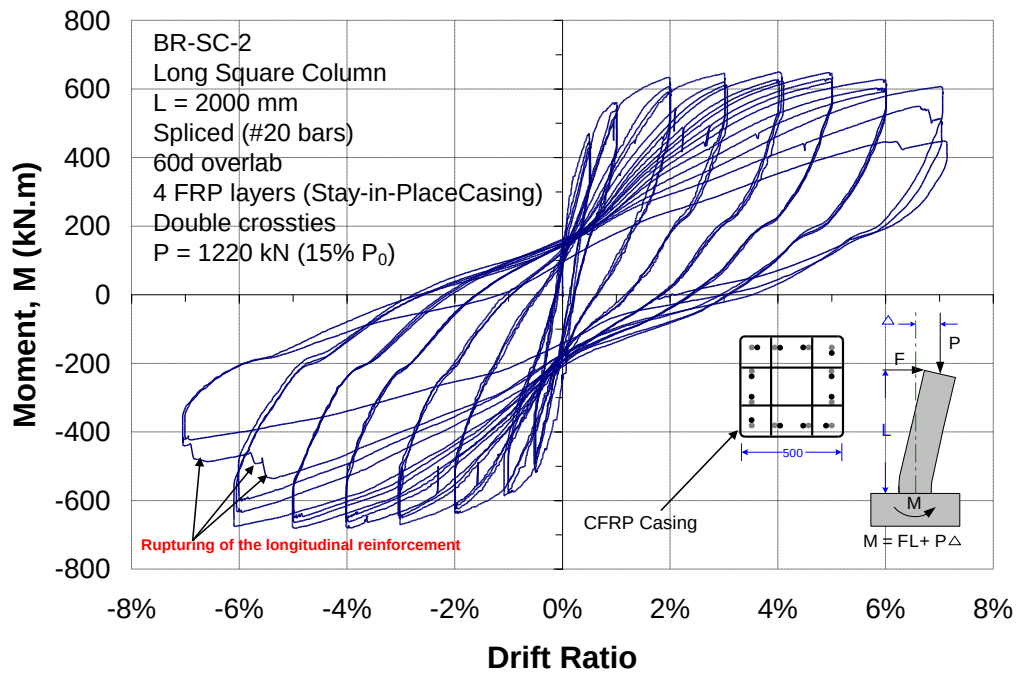
(j)

**Fig 3.47(Cont'd)** Behavior of column BR-SC-2 at different drift levels



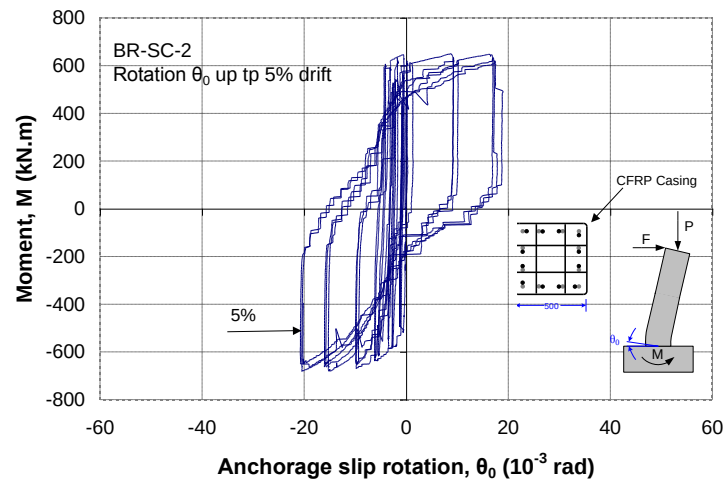
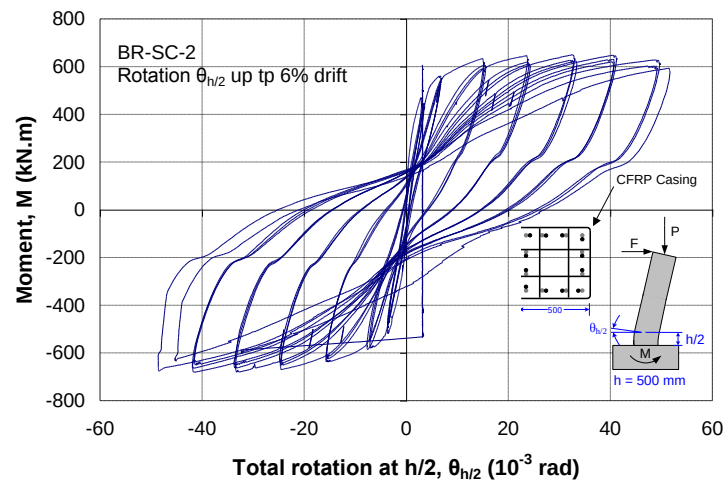
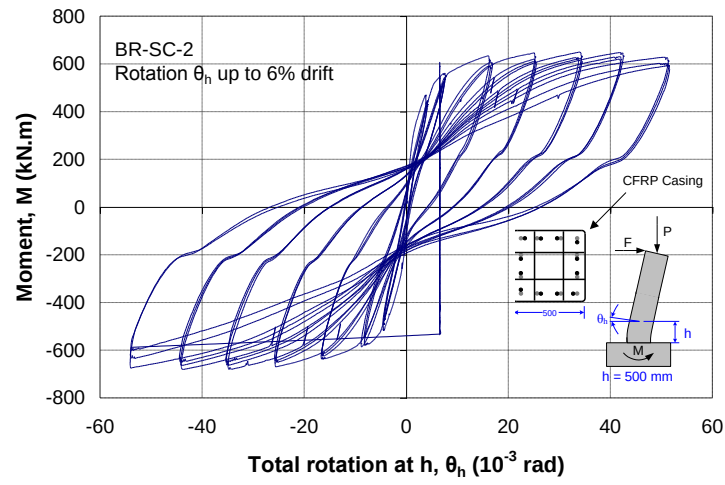


a) Force-Drift relationship



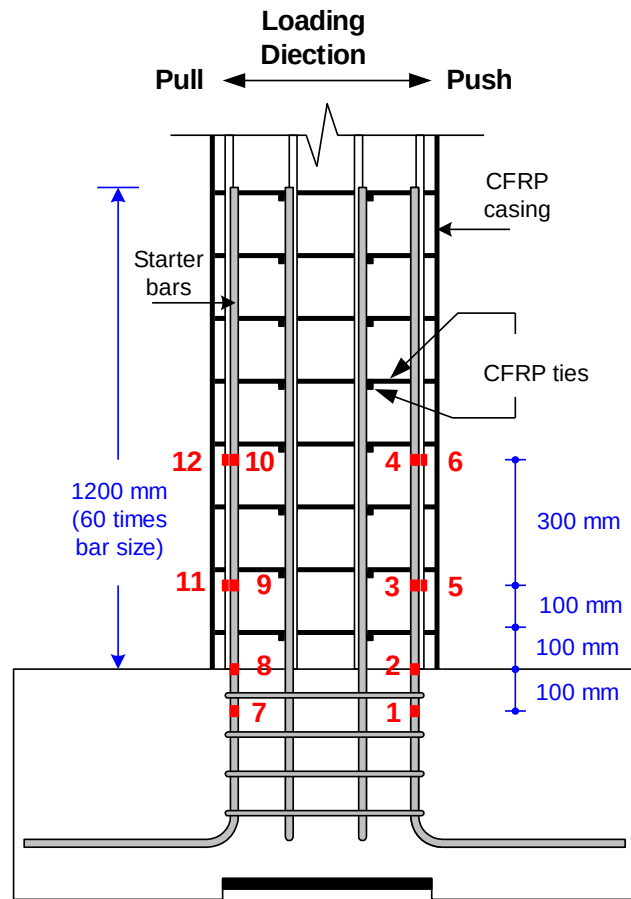
b) Moment-Drift relationship

**Fig. 3.48** Hysteretic relationship for column BR-SC-2

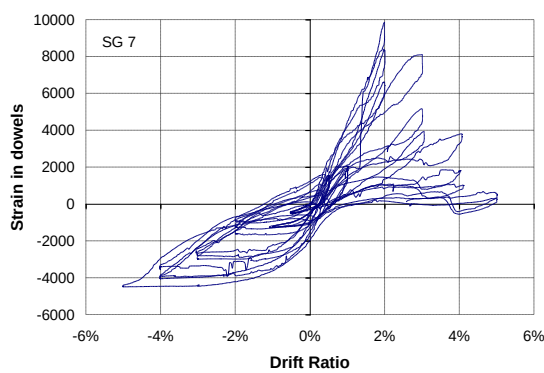


**Fig. 3.49** Rotation of the hinging region for column BR-SC-2

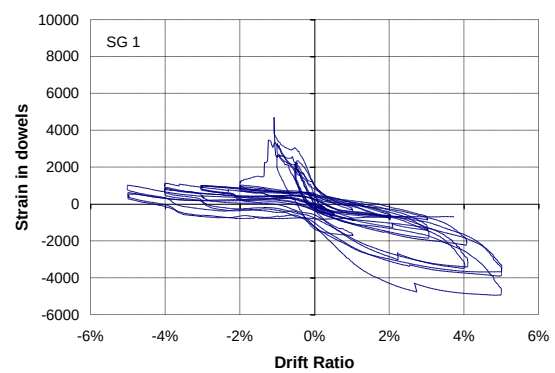




Strain gauge locations in reinforcement steel

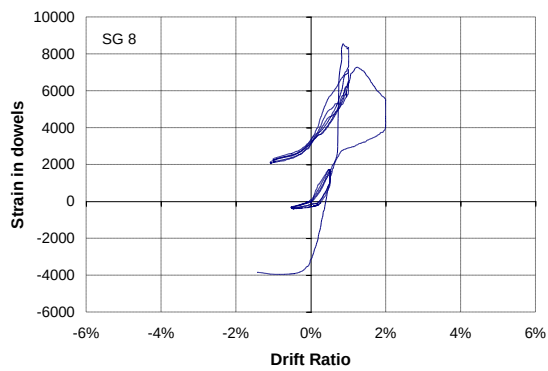


Strain gauge no. 7

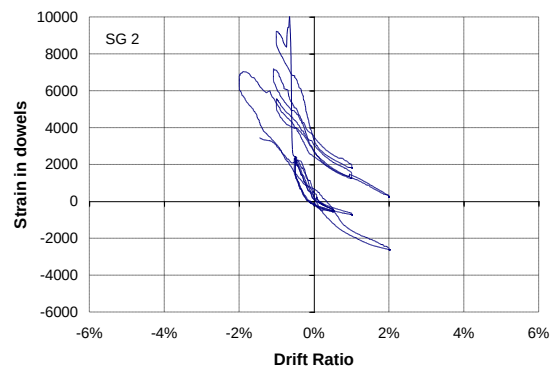


Strain Gauge no. 1

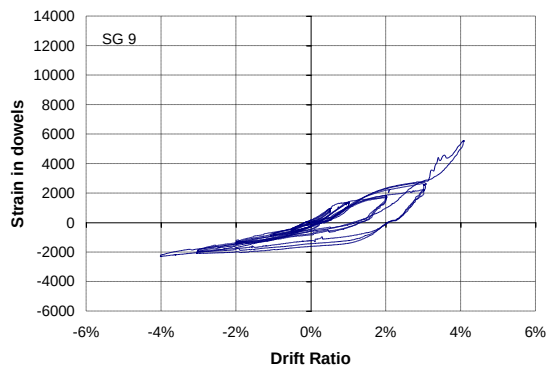
**Fig. 3.50** Dowel strains for column BR-SC-2



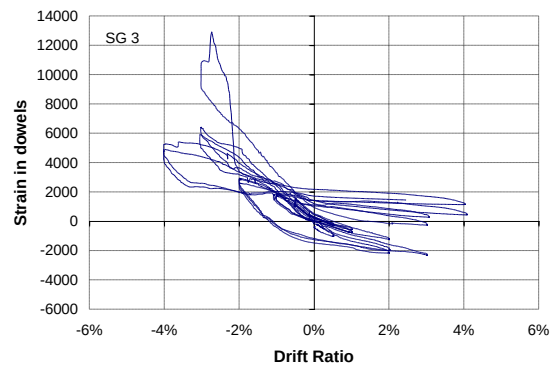
Strain gauge no.8



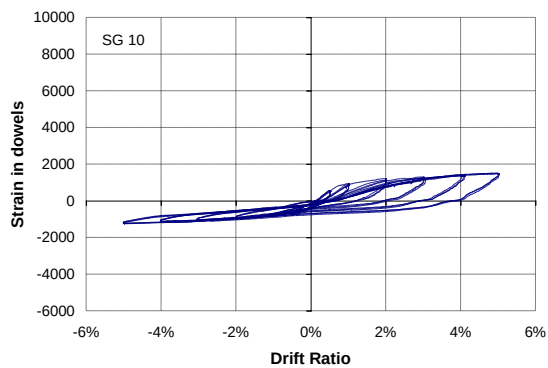
Strain gauge no.2



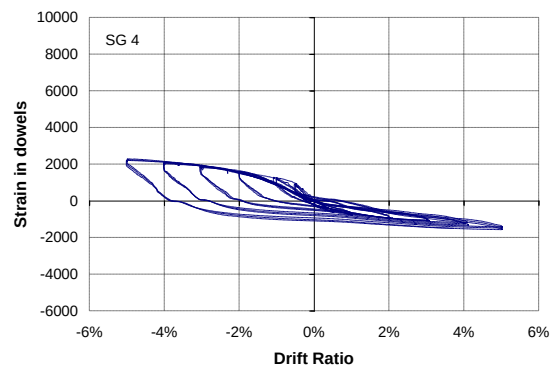
Strain gauge no.9



Strain gauge no.3

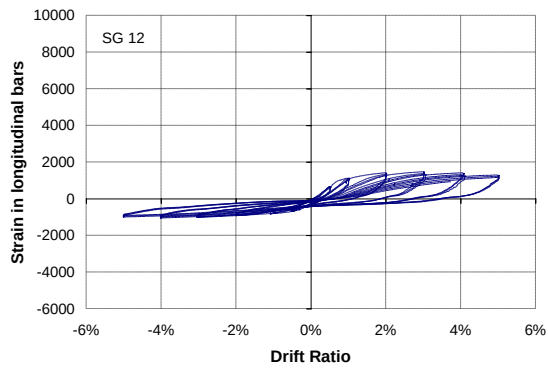


Strain gauge no.10

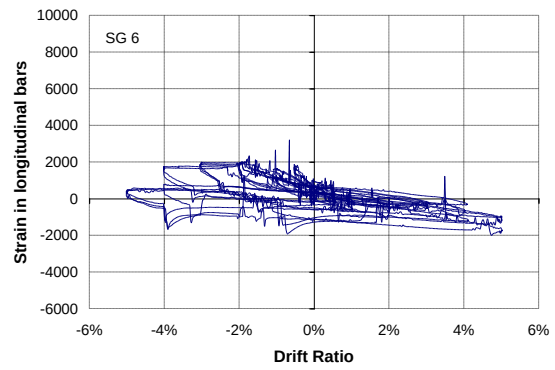


Strain gauge no.4

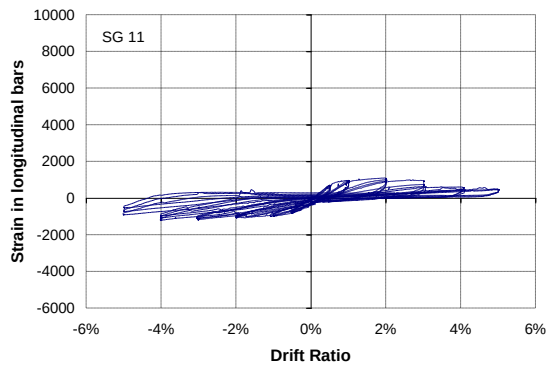
**Fig. 3.50(Cont'd)** Dowel strains for column BR-SC-2



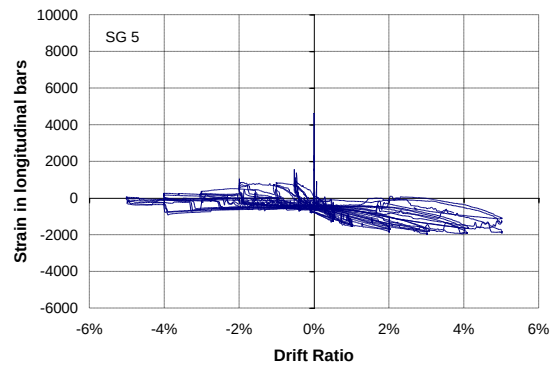
Strain gauge no. 12



Strain gauge no. 6

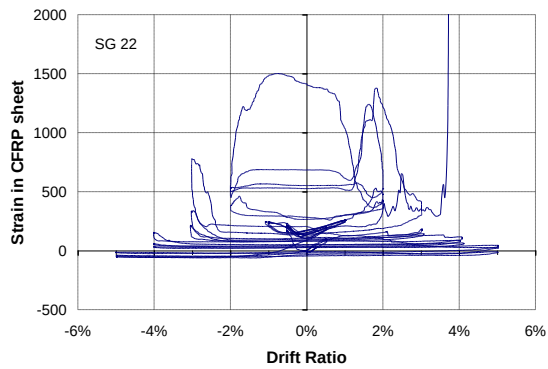
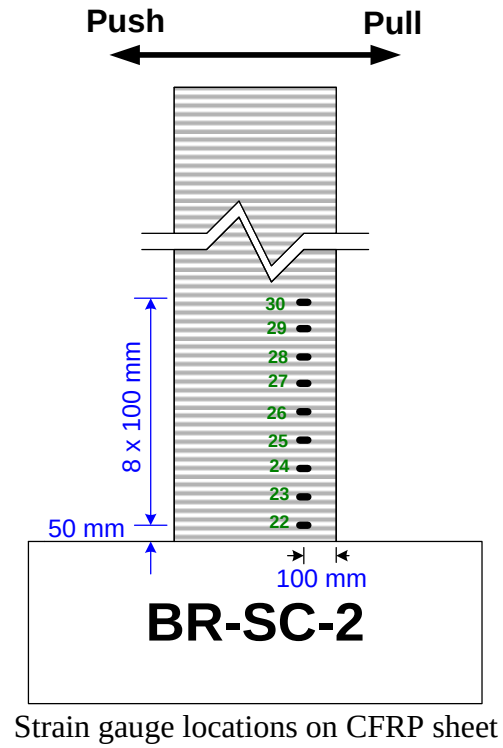


Strain gauge no. 11

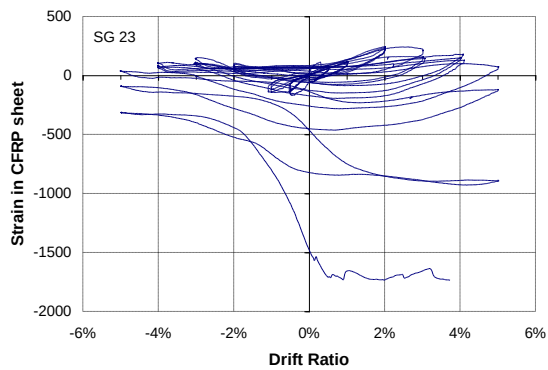


Strain gauge no. 5

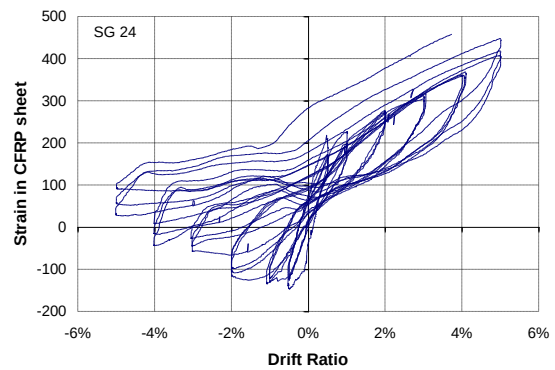
**Fig. 3.51** Longitudinal reinforcement steel strains for column BR-SC-2



Strain gauge no. 22

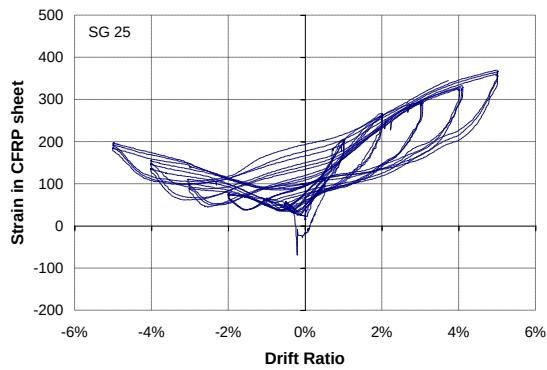


Strain gauge no. 23

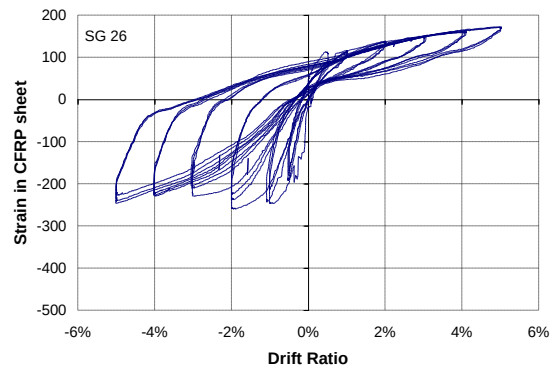


Strain gauge no. 24

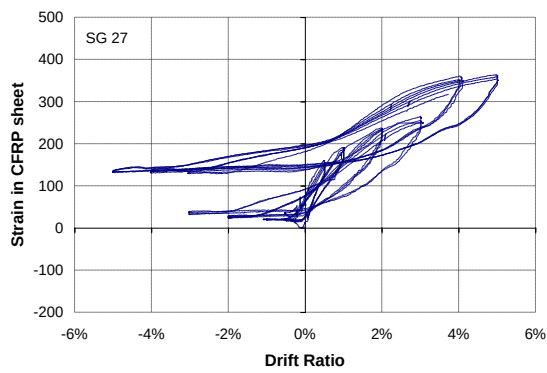
**Fig 3.52** CFRP sheet strains for column BR-SC-2



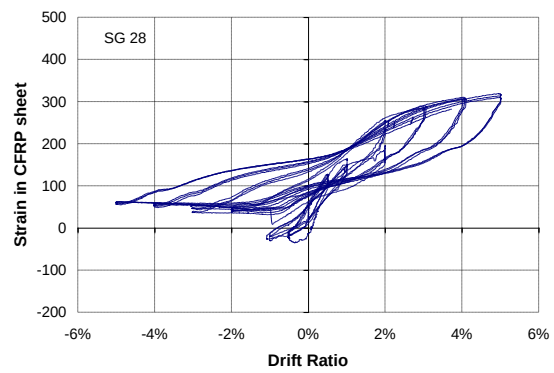
Strain gauge no. 25



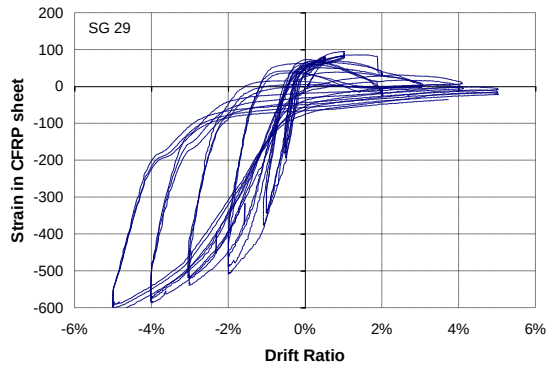
Strain gauge no. 26



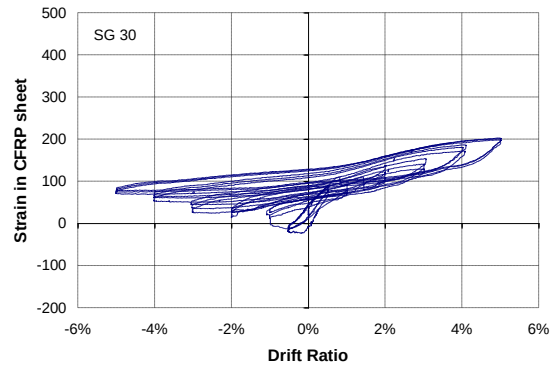
Strain gauge no. 27



Strain gauge no. 28

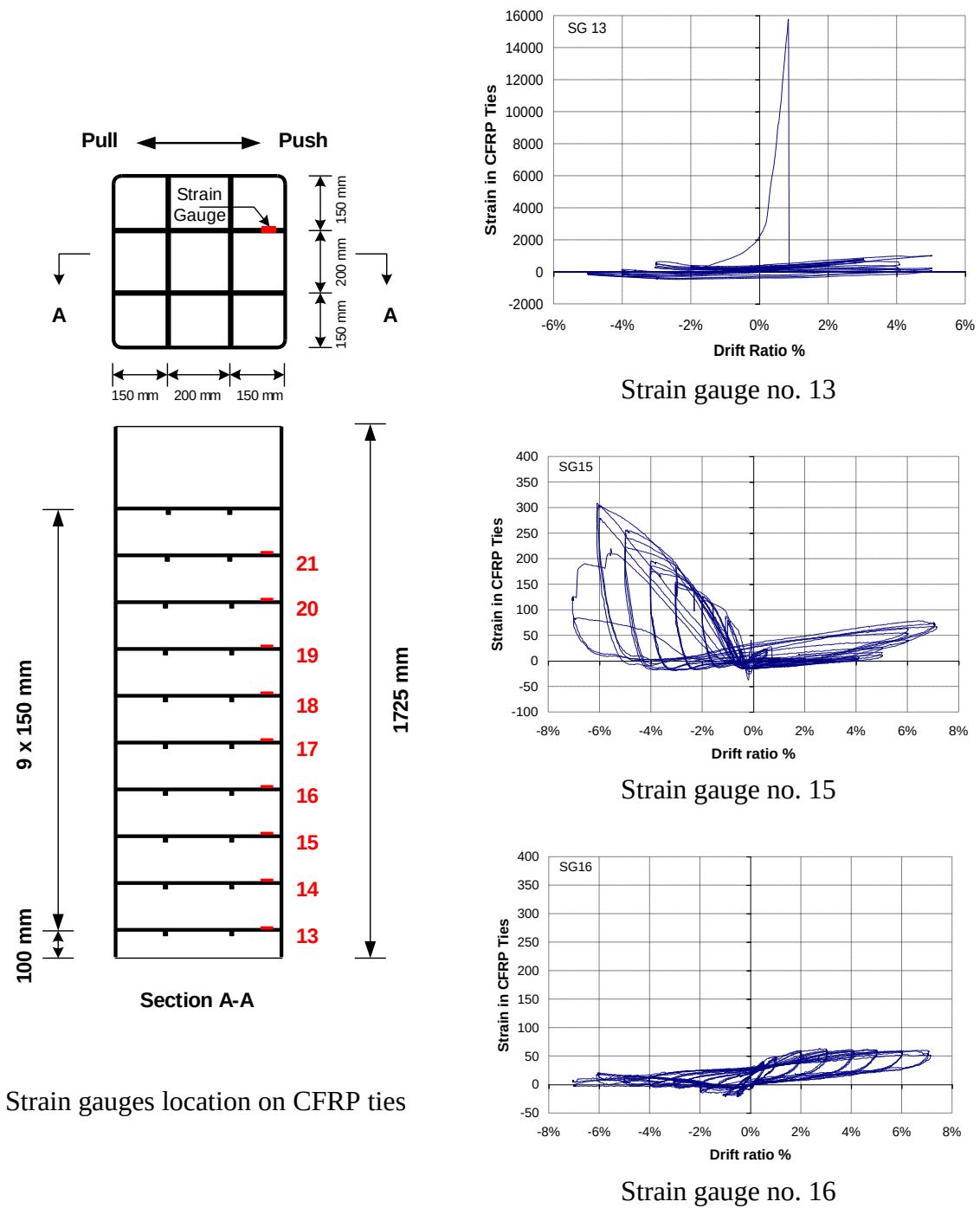


Strain gauge no. 29

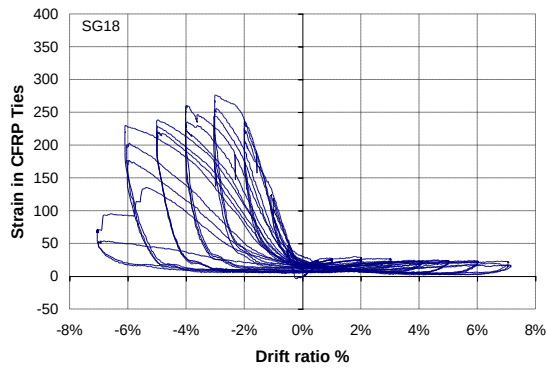


Strain gauge no. 30

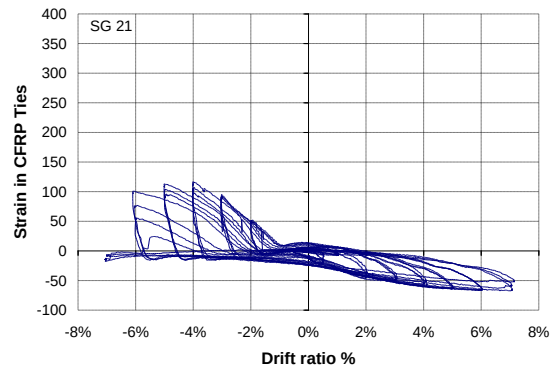
**Fig 3.52 (Cont'd)** CFRP sheet strains for column BR-SC-2



**Fig. 3.53** Strain in CFRP cross-ties for column BR-SC-2

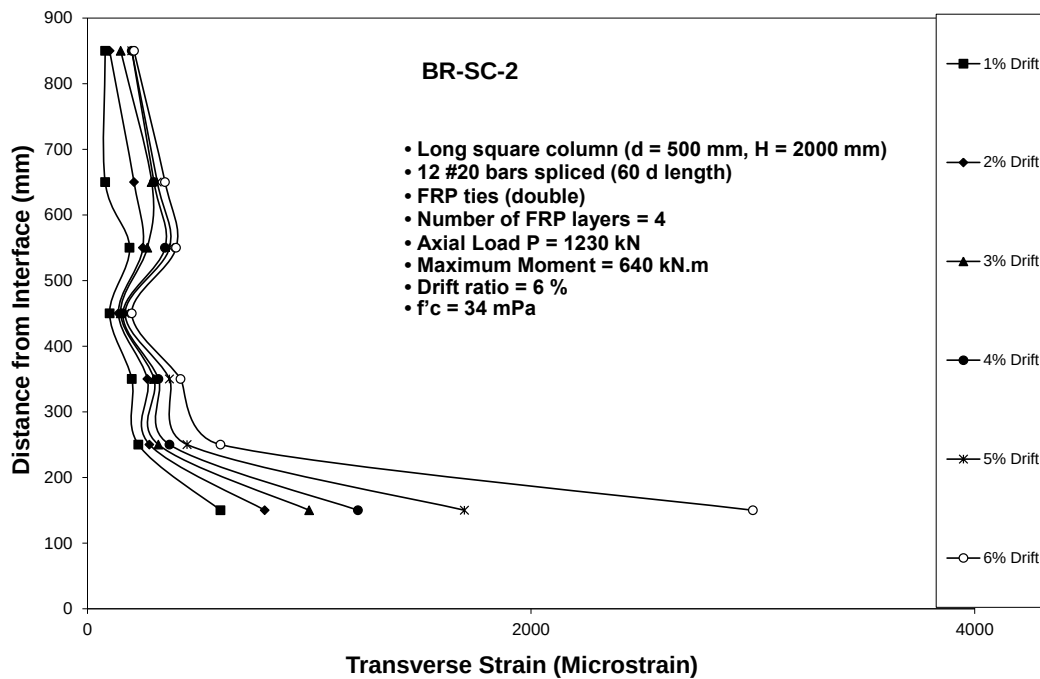


Strain gauge no. 18



Strain gauge no. 21

**Fig. 3.53(Cont'd)** Strain in CFRP crossties for column BR-SC-2



**Fig. 3.54** Fiber Strain distribution along column BR-SC-2



(a)



(b)

**Fig 3.55** BR-SC-2 after test with CFRP sheets removed





(c)



(d)

**Fig 3.55(Cont'd) BR-SC-2 after test with CFRP sheets removed**



(e)



(f) Rupturing of cross-ties

**Fig 3.55(Cont'd)** BR-SC-2 after test with CFRP sheets removed

# CHAPTER 4

## COMPARISONS AND EVALUATION OF TEST RESULTS

### 4.1 GENERAL

Behaviour of seven bridge columns tested as part of the current investigation is presented in the previous chapter. In this chapter, the seismic behaviour of as-built, seismically deficient (control columns) columns tested by others in different phases of the column retrofit research program at the University of Ottawa, columns tested earlier by the author (as part of his MEng Project) involving FRP wrapping of columns and those tested in the current research project are compared. The comparison provides an assessment of the significance of design parameters.

### 4.2 COLUMNS JACKETED WITH FRP

Comparisons of column performance are presented in the following sections for columns with and without CFRP jackets.

#### 4.2.1 LONG CIRCULAR COLUMNS WITH SPLICED BARS

Columns that are compared under this category are listed in Table 4.1. They include the control column BR-C8, retrofitted column BR-C8-2 with 4 plies of CFRP, and column BR-C8-2 with 2 plies of CFRP. The specimens are shown in Fig.4.1 and have reinforcing detailing typical of the pre-1970's design practice. The comparison emphasizes the poor seismic behaviour of RC columns with inadequate reinforcement (short lap splices and/or insufficient transverse reinforcement) in the potential plastic hinge regions and the ability of the proposed retrofitting technique to enhance this behaviour.

**Table 4.1** Long circular columns with spliced bars

| COLUMN  | SIZE   | SHEAR SPAN | RETROFITTING TECHNIQUE | # OF CFRP LAYERS | LONGITUDINAL REINFORCEMENT |
|---------|--------|------------|------------------------|------------------|----------------------------|
| BR-C8   | 508 mm | 2 m        | Control column         | -                | 12 No.20 Spliced           |
| BR-C8-1 | 508 mm | 2 m        | wrapped                | 4                | 12 No.20 Spliced           |
| BR-C8-2 | 508 mm | 2 m        | wrapped                | 2                | 12No.20 Spliced            |

**BR-C8 vs BR-C8-2**

The control column BR-C8 tested by Beausejour (2000) had the same reinforcement details as Specimen BR-C8-2. The cross-sectional diameter is 508 mm and the shear span is 2.0m. The longitudinal reinforcement consists of 12 No. 20 bars, distributed evenly along the perimeter. The bars are spliced at the bottom with 20 bar diameter splice length (490mm) as shown in Fig. 4.1. Moment-drift hysteretic relationships of columns are shown in Fig. 4.2. From the graphs it is evident that the drift capacity of the control column is 1% with significant strength decay at 2% lateral drift due to the slippage of longitudinal reinforcement. The lap length was not sufficient to develop the bond needed to prevent slippage, and the hoops were unable to clamp the vertical bars, resulting in major cracks and consequent spalling of the cover concrete in the control column, followed by the deterioration of core concrete as illustrated in Fig. 4.1a and 4.1b. On the other hand the flexural capacity of the retrofitted column was maintained up to 4% drift without much strength decay and up to 6% with 20% strength decay. The specimen BR-C8-2 was significantly more ductile due to the confinement of concrete within the plastic hinge at the bottom of the column. The flexural capacity increased by 20% as compared to the control column. The retrofit strategy delayed debonding of lap splices from 1% to 4% lateral drift. The energy dissipation was significantly improved resulting in yielding and plastic deformation of critical areas within the column, even though some pinching of hysteresis loops are evident in the retrofitted column due to the slippage of longitudinal reinforcement.

### **BR-C8-1 vs BR-C8-2**

The properties of Columns BR-C8-1 and BR-C8-2 are summarized in Table 4.1, with the former column having 4 plies of CFRP and the latter column having 2 plies only. Fig. 4.3 shows the hysteretic relationships of both columns. Column BR-C8-1 was designed following the manufacturer's recommendation of using 4 plies. From the hysteretic relationships it can be seen that Column BR-C8-1 has a stable hysteretic pattern with almost constant flexural capacity up to 6% drift, before experiencing some strength decay. There is no pinching of hysteresis loops, indicating proper clamping by the CFRP jacket in the plastic hinge region. No sign of buckling was observed in the column. The behaviour of column BR-C8-2, with reduced number of plies (two plies) was similar to that of BR-C8-1 up to 4% drift ratio. However, the hysteresis loops exhibited some pinching, signifying slight slippage of lap spliced reinforcement beyond 2% drift ratio, as shown in Fig. 4.3b. The flexural capacity was the same within this range (both had a peak flexural capacity of 520kN.m). At 5% drift, the sudden drop in the strength indicates buckling of the longitudinal reinforcement in BR-C8-2. The maximum fiber strain recorded in BR-C8-1 was  $7500\mu\epsilon$  compared to  $11000\mu\epsilon$  in specimen BR-C8-2. This is illustrated in Fig. 4.4. This strain is close to the CFRP rupturing strain of  $12000\mu\epsilon$  obtained through coupon tests. Both specimens had a satisfactory behaviour in post elastic response, with tests continuing up to 7% drift in the case of BR-C8-1 and 5% drift in the case of BR-C8-2 with little strength decay. Moment-Drift envelopes of the three columns, shown in Fig. 4.5, illustrate the amount of enhancement in the seismic behaviour through CFRP wrapping. The retrofitting was capable of providing sufficient clamping pressure to prevent complete loss of bond between the bars due to inadequate lap splice length.

### **4.2.2 SHORT CIRCULAR COLUMNS WITH SPLICED BARS**

The three columns included in this comparison are the control column BR-C10 and the retrofitted specimens BR-C10-1 and BR-C10-2 with reinforcing detailing typical of pre-

1970's design practice. The specimens are shown in Fig.4.6 with their properties summarized in Table 4.2.

**Table 4.2** Short circular columns with spliced bars

| COLUMN   | SIZE   | SHEAR SPAN | RETROFITTING TECHNIQUE | # OF CFRP LAYERS | LONGITUDINAL REINFORCEMENT |
|----------|--------|------------|------------------------|------------------|----------------------------|
| BR-C10   | 610 mm | 1.5 m      | Control column         | -                | 12 No.20 Spliced           |
| BR-C10-1 | 610 mm | 1.5 m      | wrapped                | 4                | 12 No.20 Spliced           |
| BR-C10-2 | 610 mm | 1.5m       | wrapped                | 2                | 12No.20 Spliced            |

### **BR-C10 vs BR-C10-2**

The control column BR-C10 tested by Beausejour (2000) had the same reinforcing details as for Specimen BR-C10-2 with CFRP wrapping. The cross-sectional diameter was 610 mm and the shear span was 1.5 m. The reinforcement consisted of 12 # 20 bars distributed evenly along the perimeter. The bars were spliced at the bottom, with 20 bar diameter splice length (490mm), as shown in Fig. 4.6. Moment-drift hysteretic relationships for both columns are shown in Fig.4.7. The photographs shown in Fig. 4.6 indicate that the control column was severely damaged with a drift capacity of 1%, as also illustrated in the hysteretic relationship. The failure was brittle and sudden with rapid strength decay at 2% drift ratio. Retrofitting the column considerably increased the energy dissipation allowing for a ductile behavior with gradual strength degradation and a drift capacity of 4%. The column showed increased bar slippage beyond 3% drift but maintained its strength, as the FRP jacket was able to protect the cover concrete from spalling. The retrofit did not increase the strength or the stiffness.

### **BR-C10-1 vs BR-C10-2**

Specimen BR-C10-2 was identical to BR-C10-1 except for the number of CFRP plies used in retrofitting. BR-C10-1 was wrapped with 4 plies and BR-C10-2 was wrapped with 2 plies of CFRP sheets. Both columns developed 4% lateral drift with little strength decay and experienced approximately 80% strength decay at 5% drift ratio. However, it

can be observed from the hysteretic relationships shown in Fig. 4.8 that BR-C10-1 did not experience any significant pinching until after 4% drift, which indicates some slippage of longitudinal reinforcement within the lap length beyond this deformation level. In contrast, BR-C10-2 with half as much CFRP sheets developed the slippage of reinforcement starting at 2% drift ratio. The Maximum fiber strain for BR-C10-1 was  $5300\mu\epsilon$ , compared to  $7500\mu\epsilon$  for BR-C10-2, as illustrated in Fig. 4.9.

The Moment-Drift envelopes for the three columns are plotted in Fig 4.10. The improvement in seismic behaviour of retrofitted columns is evident. The test data collected by testing columns with different number of CFRP plies lead to a better understanding of column response in establishing the design procedure presented in Chapter 5.

#### 4.2.3 LONG CIRCULAR COLUMNS WITH CONTINUOUS BARS

##### BR-C6 vs BR-C6-1

Two flexure dominant columns, with 2.0m shear span and continuous longitudinal reinforcement (without splicing), one with two plies of CFRP wrapping and the other without retrofitting are compared in this section. The specimen properties are summarized in Table 4.3.

**Table 4.3** Long circular columns with continuous bars

| COLUMN  | SIZE   | SHEAR SPAN | RETROFITTING TECHNIQUE | # OF CFRP LAYERS | LONGITUDINAL REINFORCEMENT |
|---------|--------|------------|------------------------|------------------|----------------------------|
| BR-C6   | 610 mm | 2 m        | Control column         | -                | 12 No.20 Continues         |
| BR-C6-1 | 610 mm | 2 m        | wrapped                | 2                | 12 No.20 Continues         |

The hysteretic relationships for the control Specimen BR-C6 (unretrofitted) and retrofitted column BR-C6-1 are shown in Fig 4.11. The two columns after testing are shown in Fig. 4.12. The control column, tested by Mes (1999), represents the case of continuous bars with widely spaced transverse reinforcement. The column reached 2%

drift ratio before failing in a sudden and brittle manner during the second cycle at 3% drift due to the buckling of longitudinal reinforcement (Fig.4.11a). The widely spaced hoops were unable to provide sufficient lateral restraint to prevent the buckling of compression bars and consequent deterioration and integrity of the column. The hysteretic relationship for the retrofitted column indicates a very significant improvement (Fig. 4.11b). The specimen behaved in a superior manner with increased strength until 5% drift ratio before starting to loose some of this strength, albeit very small. No doubt that the CFRP wrapping was able to maintain the integrity of the column effectively, supporting reinforcement against buckling and confining concrete for improved deformability, increasing lateral drift capacity up to 8% when the stroke limit of the horizontal actuator was exceeded. The Moment-Drift envelopes of the two columns are shown in Fig. 4.13. The retrofitting strategy considerably increased the ductility and energy dissipation capacity of the column.

#### 4.2.4 SHORT SQUARE COLUMNS WITH CONTINUOUS BARS

##### BR-S1 vs BR-SS-R

Two shear dominant (short shear span) columns, with continuous longitudinal reinforcement (no splice deficiency) are compared to assess the effectiveness of the retrofit method employed. The specimens are shown in Fig.4.14 with their properties summarized in Table 4.4.

**Table 4.4** Short square columns with continues bars

| COLUMN  | SIZE   | SHEAR SPAN | RETROFITTING TECHNIQUE | # OF CFRP LAYERS | LONGITUDINAL REINFORCEMENT |
|---------|--------|------------|------------------------|------------------|----------------------------|
| BR-S1   | 550 mm | 1.5 m      | Control column         | -                | 12 No.25 Continuous        |
| BR-SS-R | 500 mm | 1.5 m      | wrapped                | 3                | 12 No.20 Continuous        |

The control column BR-S1 was tested by Yalcin (1997) and differs slightly from the retrofitted column BR-SS-R with regard to the sectional dimension and bar diameter.



BR-S1 was 550 x 550 mm and reinforced with 12 #25 bars compared to 500 x 500mm and 12 #20 bars in BR-SS-R. However, the shear span and transverse reinforcement were identical. Except for the higher flexural capacity recorded in the control specimen (due to the higher percentage of flexural reinforcement), the use of this specimen to investigate the ductility enhancement in column BR-SS-R would provide a valid comparison. The hysteretic relationships for both columns are shown in Fig. 4.14. As depicted, BR-S1 failed during the second cycle at 2% drift due to the buckling of longitudinal reinforcement. The drift capacity was 1% prior to failure. The performance of the retrofitted column (BR-SS-R) was significantly better, with column deformability increased up to 6% before failure at the second cycle of 7% drift due to the rupturing of longitudinal reinforcement.

#### **4.2.5 SUMMARY OF COMPARISONS FOR WRAPPED COLUMNS**

Bridge columns with poor reinforcement and seismic detailing tested under simulated seismic loading are compared with those retrofitted and tested with CFRP jackets to investigate the effectiveness of the retrofit technique in enhancing strength and deformability of columns. The comparison includes previously tested control columns and columns retrofitted using the same technique with different number of CFRP plies (Elnabelsy, 2000).

The following conclusions can be drawn from the comparisons of column tests reported above:

- Columns confined with CFRP sheets can develop ductile behaviour under simulated seismic loading, exceeding significantly the ductility capacity of 4.0 ( $R_d = 4.0$ ) implied in the current National Building Code of Canada (NBCC 2010) for fully ductile columns if they do not suffer from splice deficiency. Circular columns with splice deficiencies also benefit from CFRP jacketing when designed properly, as described in Chapter 5.
- Unlike conventional steel reinforcement that confines the core concrete, CFRP wrapping confines the entire column section, including the cover concrete, and

functions as continuous transverse reinforcement, maintaining the integrity of the entire column.

- Circular columns wrapped with CFRP sheets exhibits improved confinement action caused by hoop tension and allow higher level of plastification and energy dissipation capacity.
- While wrapped square columns with continuous reinforcement showed significant improvement in deformability, more research is needed to investigate the relationships between the parameters of FRP jacketing and the characteristics of longitudinal reinforcement splice region.
- CFRP jackets are highly effective in controlling diagonal tension cracking under shear force reversals, improving shear capacity in shear-dominant columns, and changing the behavior from brittle shear failure to ductile flexural response.

### **4.3 COLUMNS CAST IN STAY-IN-PLACE FORMWORK**

The scope of current research includes the use of CFRP transverse reinforcement for new concrete columns in the form of prefabricated CFRP tubes. CFRP tubes confine concrete, provide shear resistance, and protect the steel reinforcement against corrosion, while functioning as a stay-in-place formwork. Although this method is intended for new construction, with reinforcement splicing conforming to the current code requirements, two of the specimens tested included longitudinal bars spliced using  $20d_b$  splice length (which is less than that required in the current code) to explore the possibility of using reduced splice lengths in well confined FRP casings. One potential difference between the existing columns wrapped with CFRP to form a column jacket and the new columns encased in CFRP tubes as formwork is the active confinement pressure that can build-up during concrete casting. The formwork pressure generated by fresh concrete may apply active pressure on the FRP tube, a phenomenon that is not applicable in the case of wrapped columns. However, the level of active pressure in the columns tested was measured to be very small, in comparison with the passive confinement pressure generated during testing. Therefore, in comparing these columns with wrapped columns, this difference in the construction process is neglected. The comparisons in this section

include circular columns BR-CC-1 vs BR-C8-2 and square columns BR-SC-1 vs BR-S5-1 and BR-S3 and BR-SC-2.

### 4.3.1 CIRCULAR COLUMNS

#### BR-CC-1 vs BR-C8-2

The properties of the two circular columns compared are given in Table 4.5. These two columns were virtually identical except for the wrapped column having 2 layers of CFRP, as opposed to the stay-in-place column with 3 layers of CFRP. Both specimens had 20 bar diameter (short) splice length.

**Table 4.5** long circular columns with spliced bars

| COLUMN  | SIZE   | SHEAR SPAN | RETROFITTING TECHNIQUE | # OF CFRP LAYERS | LONGITUDINAL REINFORCEMENT  |
|---------|--------|------------|------------------------|------------------|-----------------------------|
| BR-CC-1 | 508 mm | 2 m        | Stay-in-Place formwork | 3                | 12 No.20 Spliced (no hoops) |
| BR-C8-2 | 508 mm | 2 m        | wrapped                | 2                | 12 No.20 Spliced            |

The hysteretic relationships for columns BR-CC-1 and BR-C8-2 are illustrated in Fig. 4.15. The figure shows that the behaviours of these two specimens are very close in terms of load capacity and stiffness. Data for recorded strains on longitudinal reinforcement confirmed the similarity in specimen response to applied loading. From the data for the two specimens illustrated in the previous chapter, the recorded strains were almost identical for the dowels and were smaller by 15% for the longitudinal reinforcement. Strains recorded on FRP sheet were lower on the prefabricated casing. This is due to the increased jacket thickness (3 plies) as compared to the wrapped specimen that had 2 plies.

CFRP sheets in the form of prefabricated casing were able to provide a certain amount of clamping effect that resulted in a 4% drift ratio for specimen BR-CC-1. It is believed that increasing the lap splice would result in further improvements in ductility. Therefore, a

lap splice length of  $60 d_b$  was considered for future tests of circular and square columns, conforming to current practice for steel reinforced concrete columns.

#### 4.3.2 SQUARE COLUMNS

##### BR-SC-1 vs BR-S5-1

The two square columns compared in this section are shown in Fig.4.6 with their properties summarized in Table 4.6.

**Table 4.6** Long square columns with spliced bars

| COLUMN  | SIZE   | SHEAR SPAN | RETROFITTING TECHNIQUE                        | # OF CFRP LAYERS | LONGITUDINAL REINFORCEMENT |
|---------|--------|------------|-----------------------------------------------|------------------|----------------------------|
| BR-SC-1 | 500 mm | 2 m        | Stay-in-Place formwork with single cross ties | 4                | 8 #25 spliced              |
| BR-S5-1 | 500 mm | 2 m        | wrapped                                       | 5                | 12 #20 spliced             |

Specimen BR-SC-1 with prefabricated square casing and integrated crossties showed a ductile behaviour close to BR-S5-1 with wrapped CFRP sheets. Hysteretic relationships for the two columns are illustrated in Fig. 4.16. Both specimens exhibit poor ductile behaviour as the drift capacity was 2% for BR-SC-1 and 3% for BR-S5-1.

##### BR-SC-2 VS BR-S3

The results obtained from the previous two square columns discussed above clearly demonstrate once again that neither CFRP jacketing, nor the CFRP casing, even with single crossties embedded in the casing, was able to reduce the pinching of hysteresis loops or prevent the bar slippage when the splice length of longitudinal reinforcement was at  $20d_b$  (reflecting pre-1970s practice). Therefore, the splice length in column BR-SC-2 has been increased to  $60d_b$  to promote continuity in the longitudinal tension reinforcement during reversed cyclic loading. Column BR-SC-2 was built to represent

newly constructed columns using CFRP Stay-in-Place casing with integrated crossties, and a splice length of  $60d_b$  (120 cm). Due to the use of improved (longer) splice length, it is appropriate to compare this column to a specimen with continuous reinforcement. Therefore this column is compared with control column BR-S3 tested by Mes (1999) in an earlier phase of the overall research program. These specimens are shown in Fig.4.17 with their properties summarized in Table 4.7.

**Table 4.7** Long square columns with continues bars

| COLUMN  | SIZE   | SHEAR SPAN | RETROFITTING TECHNIQUE                     | # OF CFRP LAYERS | LONGITUDINAL REINFORCEMENT      |
|---------|--------|------------|--------------------------------------------|------------------|---------------------------------|
| BR-S3   | 500 mm | 2 m        | Control column                             | -                | 12 No.20 continues              |
| BR-SC-2 | 500 mm | 2 m        | Stay-in-Place casing with double crossties | 4                | 12 No.20 spliced $60 d$ overlap |

The control column BR-S3 with continuous bars had flextural dominant behaviour. The column reached maximum capacity during the first cycle at 2% drift as shown in Fig. 4.17. The longitudinal reinforcement yielded at 1% drift, and the peak effective lateral load was 285 kN correspond to a flexural capacity of 630kN.m. The column failed during the last cycle at 3% drift due to the buckling of longitudinal reinforcement. The drift capacity of the column was 2%. Fig. 4.17a shows column BR-S3 at the end of testing.

The hysteretic relationship for the companion column cast in CFRP casing with double crossties indicates very significant improvement in behaviour as illustrated in Fig. 4.17b. The specimen behaved very well with increased strength until 6% drift ratio. The specimen failed due to the rupturing of longitudinal reinforcement in bending. The failure did not initiate due to bar buckling, even at this excessive level of inelastic response. This signifies that the FRP casing and the internal FRP crossties built in with the casing maintained the integrity of the column, providing effective support to the longitudinal reinforcement in compression against buckling, while confining concrete for improved deformability. The moment-drift relationships for the two columns are shown in Fig. 4.17. After the test and after the removal of the FRP casing, it was observed that a few of the internal crossties had ruptured, showing the effectiveness of the internal crossties.

### **4.3.3 SUMMARY OF COMPARISONS FOR COLUMNS CAST IN STAY-IN-PLACE FORMWORK**

The use of CFRP tubes as stay-in-place column formwork has been investigated through large-scale column tests. The following conclusions are drawn from the test results:

- CFRP stay-in-place formwork, in the form of circular or square tubes can be used effectively as transverse reinforcement for confinement and shear. The tubes also provide sufficient clamping force against the slippage of spliced longitudinal reinforcement in plastic hinge regions of circular columns due to the superior performance of FRP in hoop tension. FRP tubes for square columns are not effective in splice deficient regions with splice length equal to  $20d_b$ .
- Flexure-dominant bridge columns in CFRP tubes, with continuous longitudinal reinforcement (or with reinforcement having sufficient splice lengths) can develop lateral drift capacities in excess of 6%. Bridge columns with short shear spans can be reinforced for shear if cast in CFRP tubes. CFRP stay-in-place formwork in such columns can suppress shear failure and promote ductile flexural response.
- The performance of reinforced concrete columns in CFRP jackets is similar to that in CFRP stay-in-place formwork.
- Stay-in-place formwork provides sufficient lateral bracing against the buckling of compression reinforcement if used with internal concrete cover, without the need for transverse steel reinforcement.
- The performance of square CFRP stay-in-place formwork can be enhanced if internal FRP crossties are embedded in the tube. Internal crossties, having at least equal area of fibres as the amount used on one side of the perimeter CFRP tube provide sufficient lateral restraints against the expansion of concrete under flexural compression, confining concrete as in the case of conventional steel crossties and overlapping hoops.



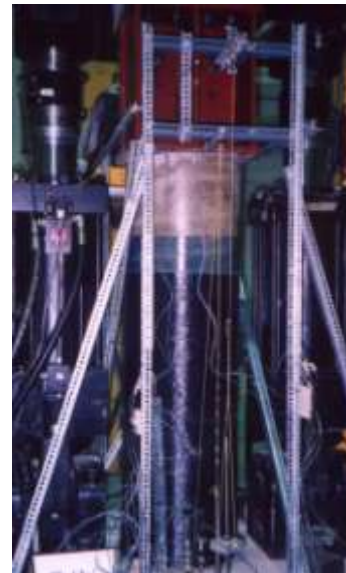
a) Control column BR-C8



b) Control column BR-C8 after test

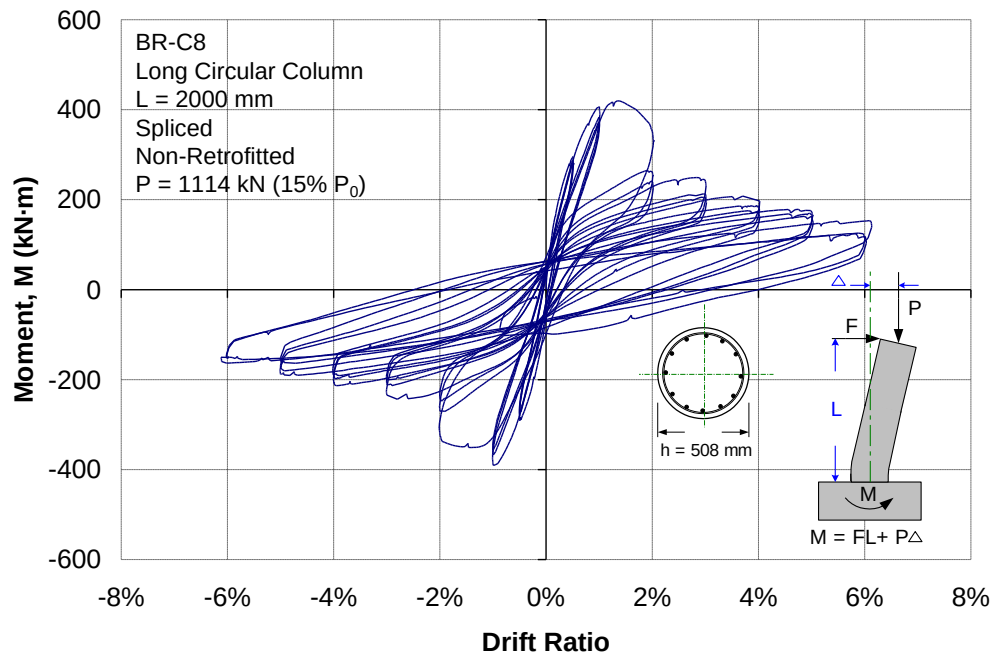


c) Retrofitted column BR-C8-1  
(4 plies of CFRP)

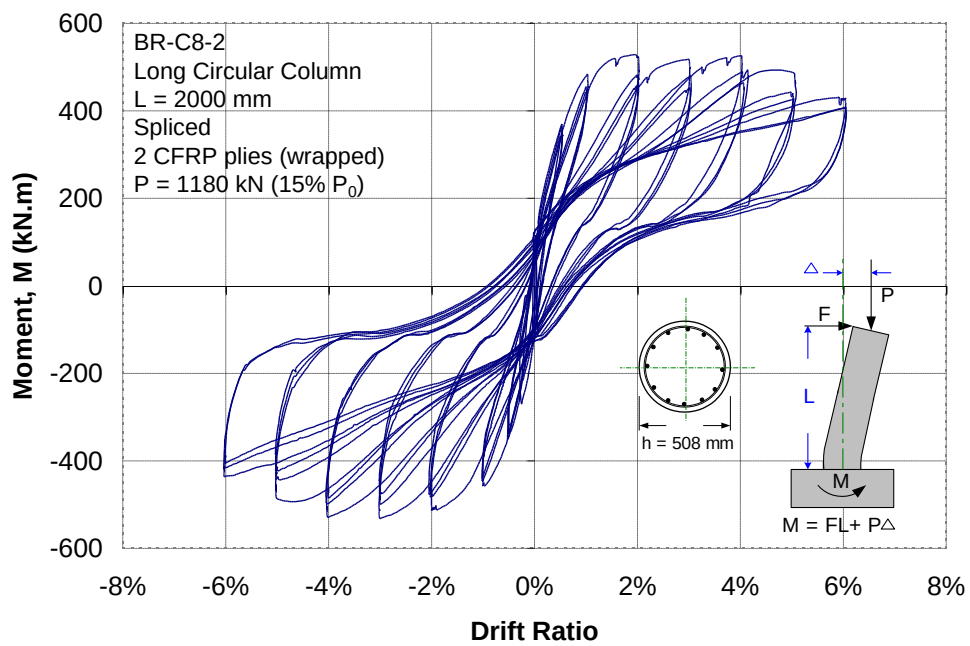


d) Retrofitted column BR-C82  
(2 plies of CFRP)

**Fig. 4.1** Long circular columns (spliced bars)



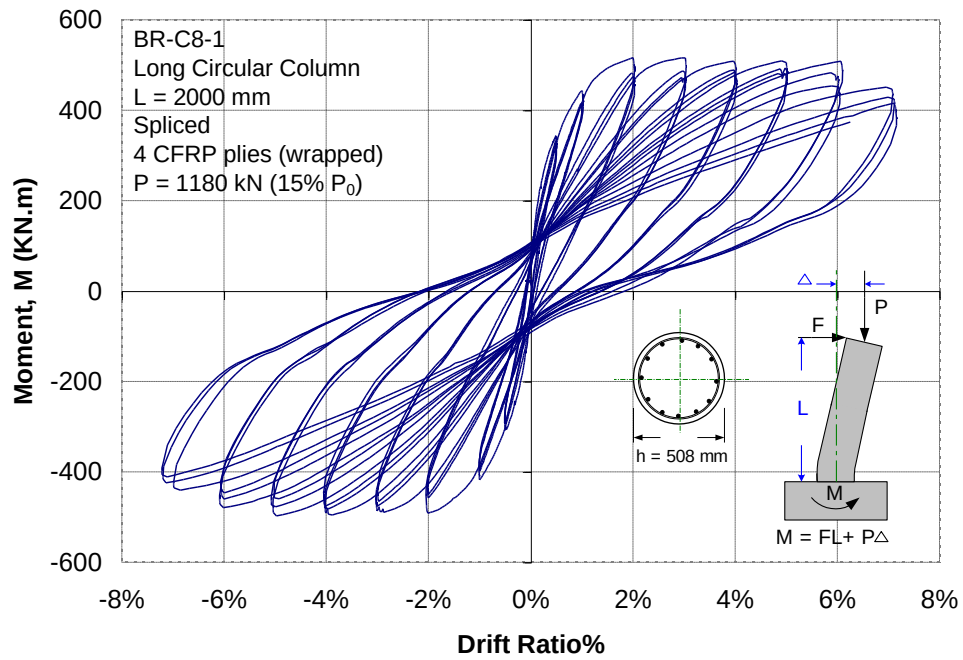
**a)** Control column BR-C8 (Beausejour, 2000)



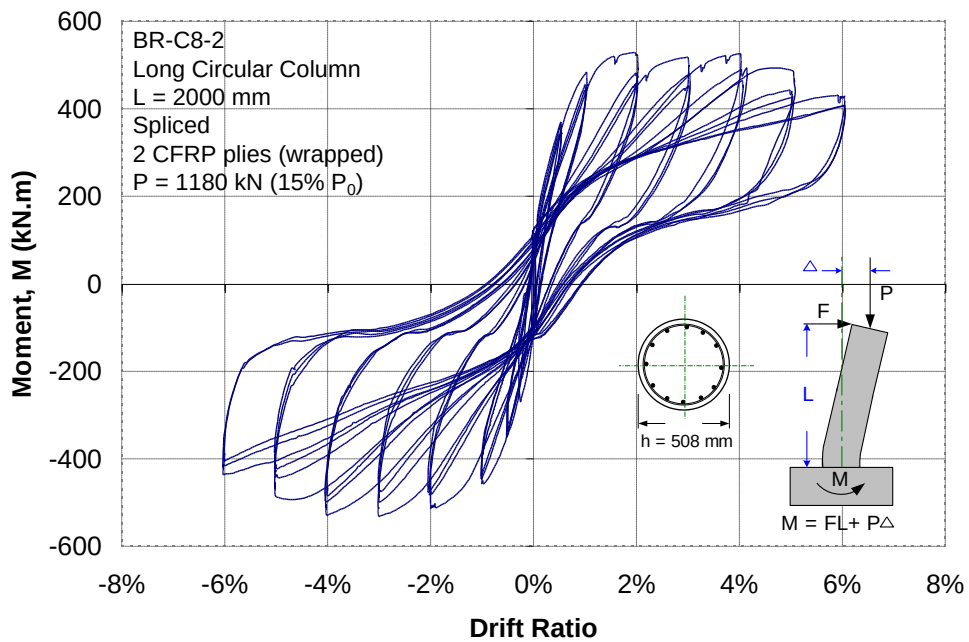
**b)** Retrofitted column BR-C8-2, 2 plies of CFRP

**Fig. 4.2** Moment-drift hysteretic relationships for control column BR-C8 and retrofitted column BR-C8-2



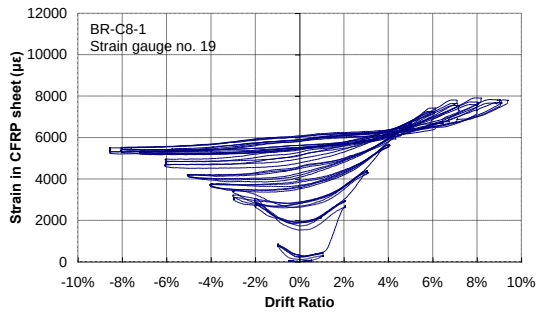


a) Retrofitted column BR-C8-1, 4 plies of CFRP, (Elnabelsy, 2000)

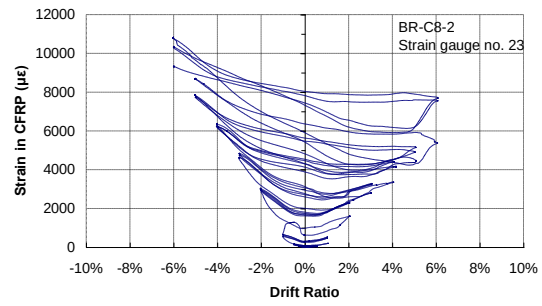


b) Retrofitted column BR-C8-2, 2 plies of CFRP

**Fig. 4.3** Moment-drift hysteretic relationships for retrofitted columns BR-C8-1 and BR-C8-2

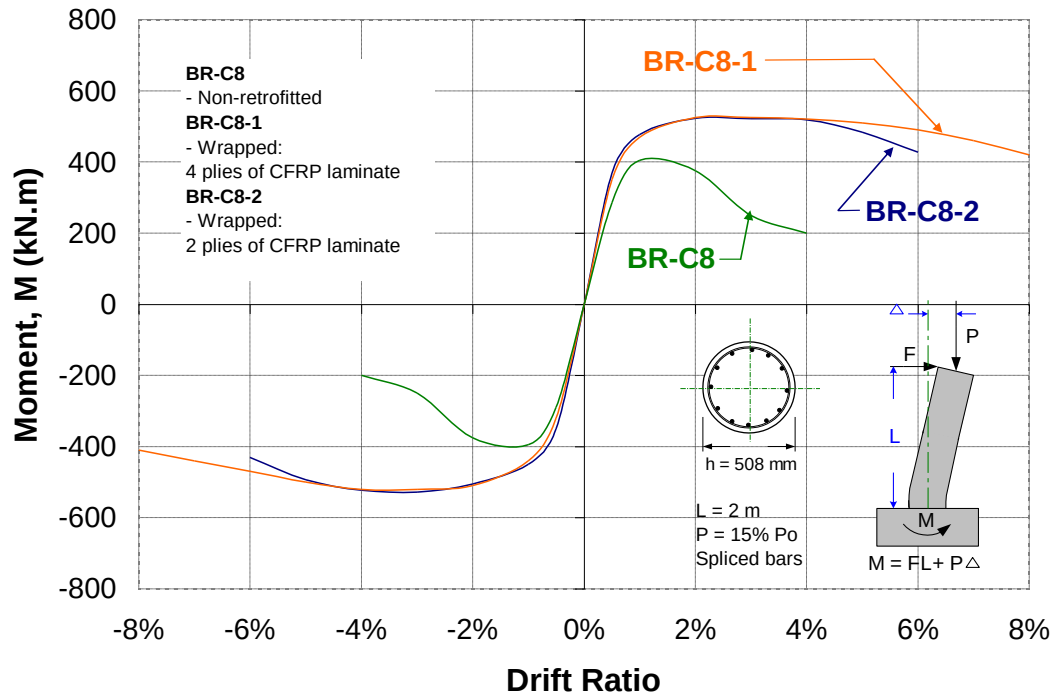


a) Maximum CFRP strains for retrofitted column BR-C8-1



a) Maximum CFRP strains for retrofitted column BR-C8-2

**Fig 4.4** Maximum fiber strains in columns BR-C8-1 & BR-C8-2



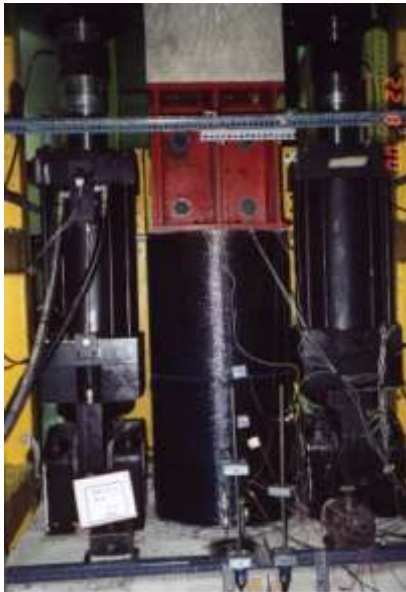
**Fig 4.5** Moment-drift envelopes for control column BR-C8 and retrofitted columns BR-C8-1 & BR-C8-2



Control column BR-C10



Control column BR-C10 after test

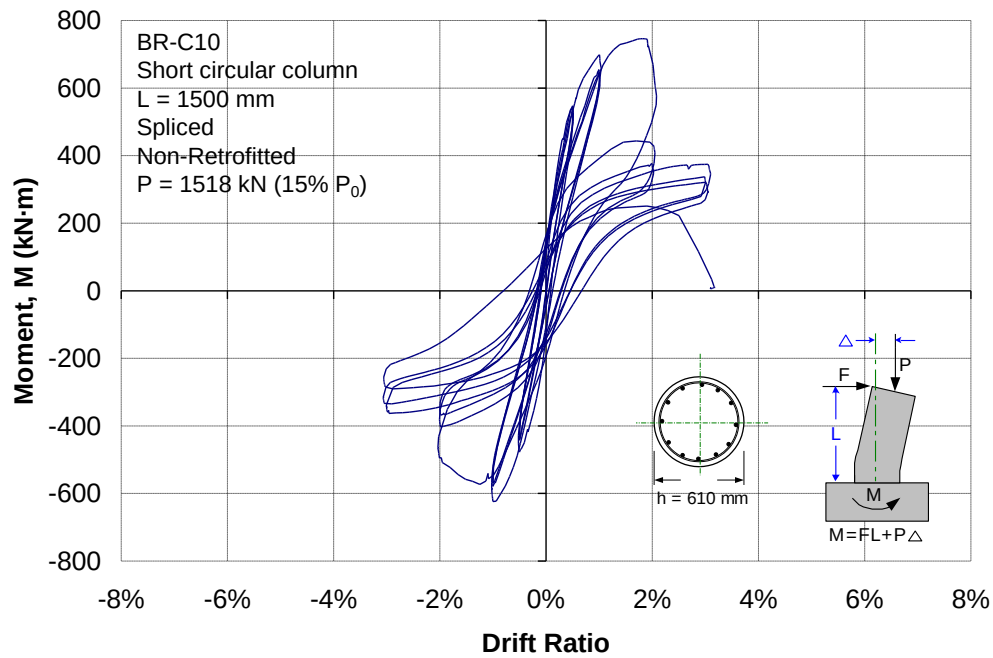


Retrofitted column BR-C10-1  
(4 plies of CFRP)

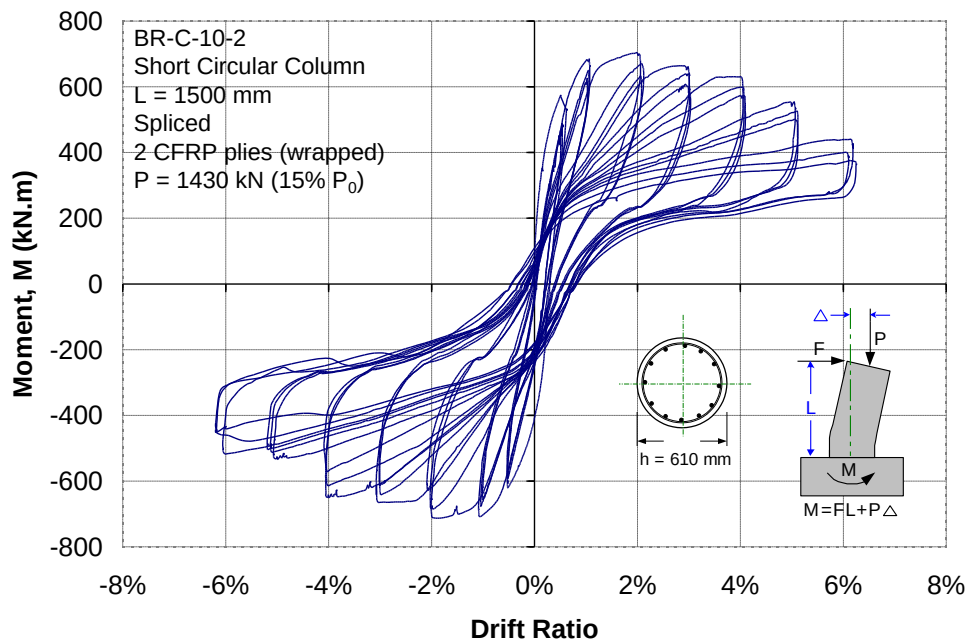


Retrofitted column BR-C10-2  
(2 plies of CFRP)

**Fig. 4.6** Short circular columns with spliced bars

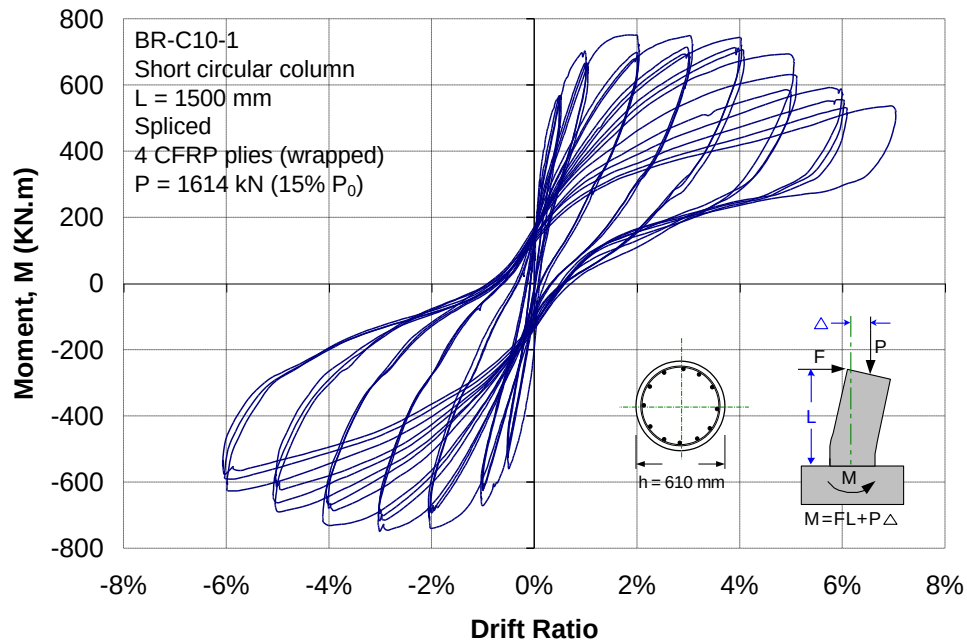


a) Control column BR-C10 (Beausejour, 2000)

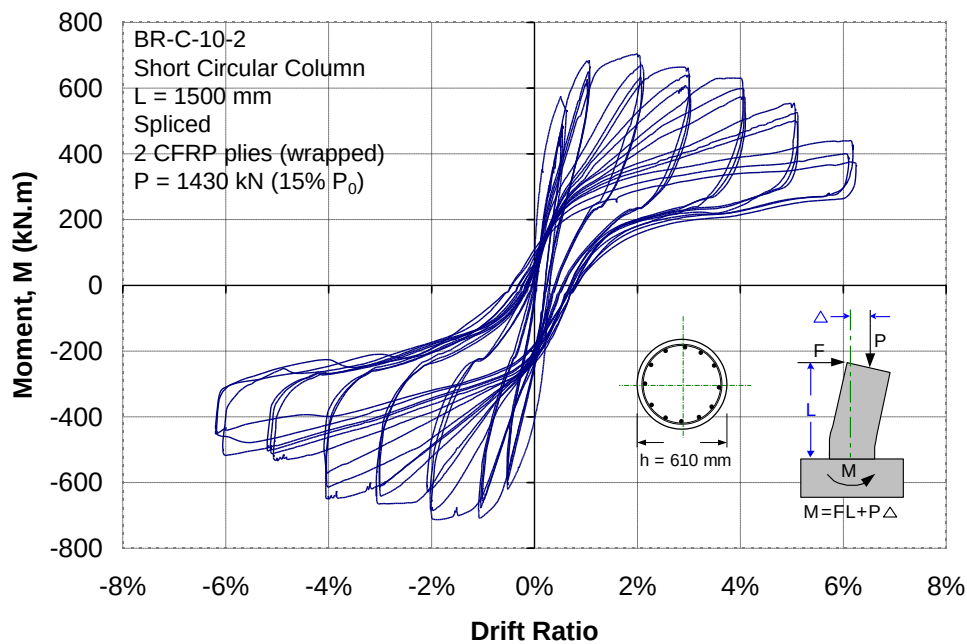


b) Retrofitted column BR-C10-2, 2 plies of CFRP

**Fig. 4.7** Moment-drift hysteretic relationships for control column BR-C10 and retrofitted column BR-C10-2

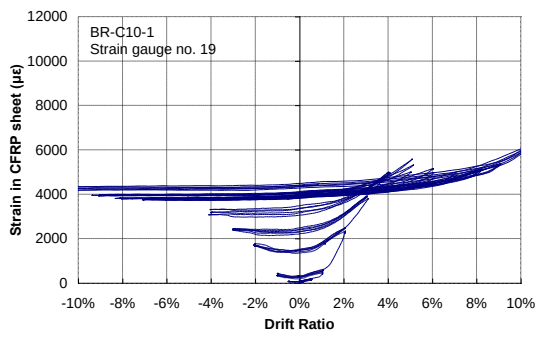


a) Retrofitted column BR-C10-1, 4 plies of CFRP, (Elnabelsy, 2000)

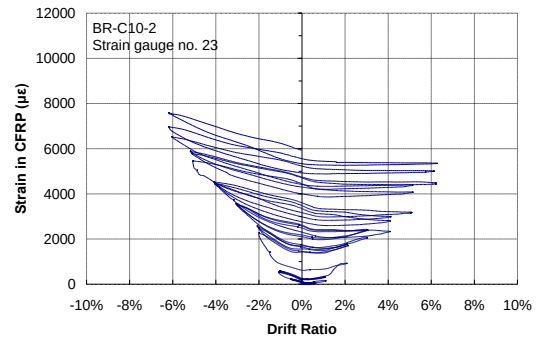


b) Retrofitted column BR-C10-2, 2 plies of CFRP

**Fig. 4.8** Moment-drift hysteretic relationships for retrofitted columns BR-C10-1 and BR-C10-2

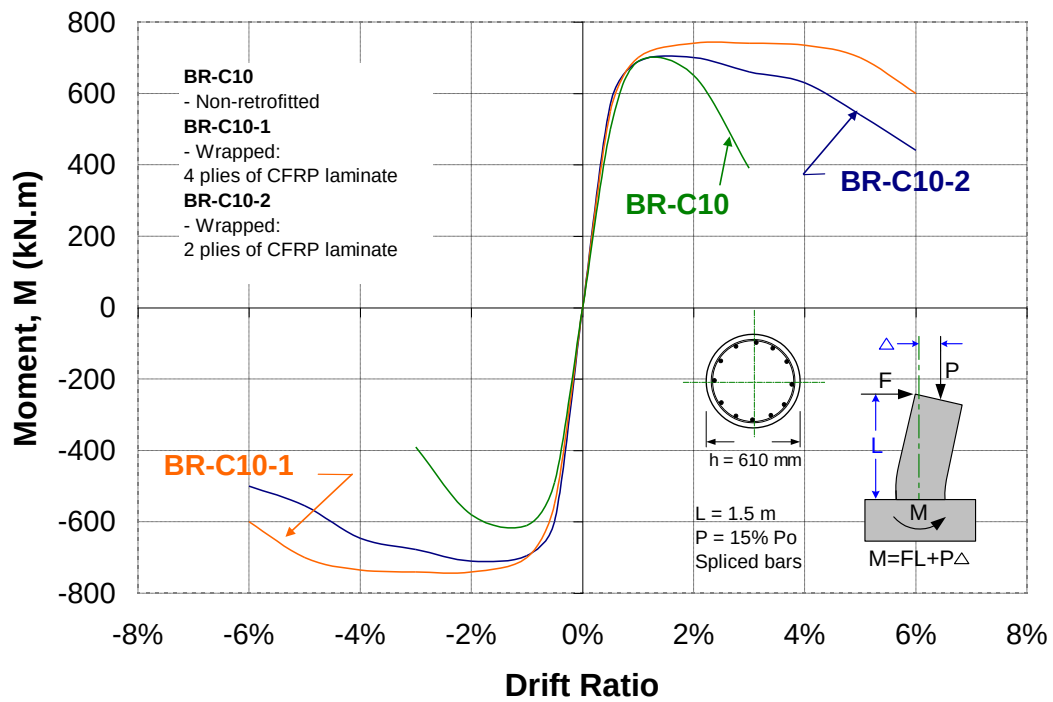


a) Maximum CFRP strains for retrofitted column BR-C10-1

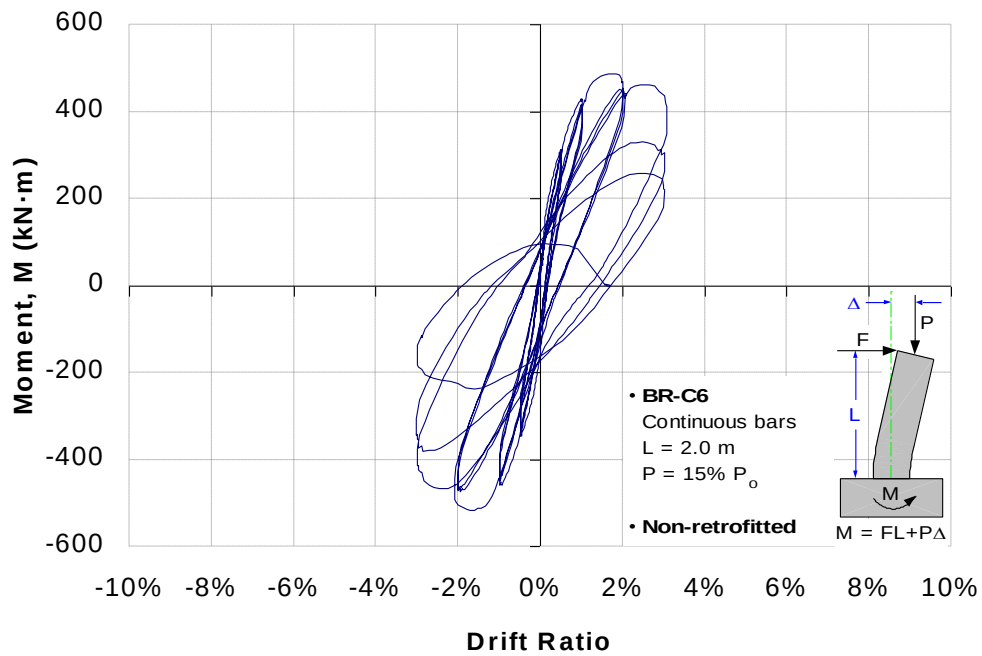


a) Maximum CFRP strains for retrofitted column BR-C10-2

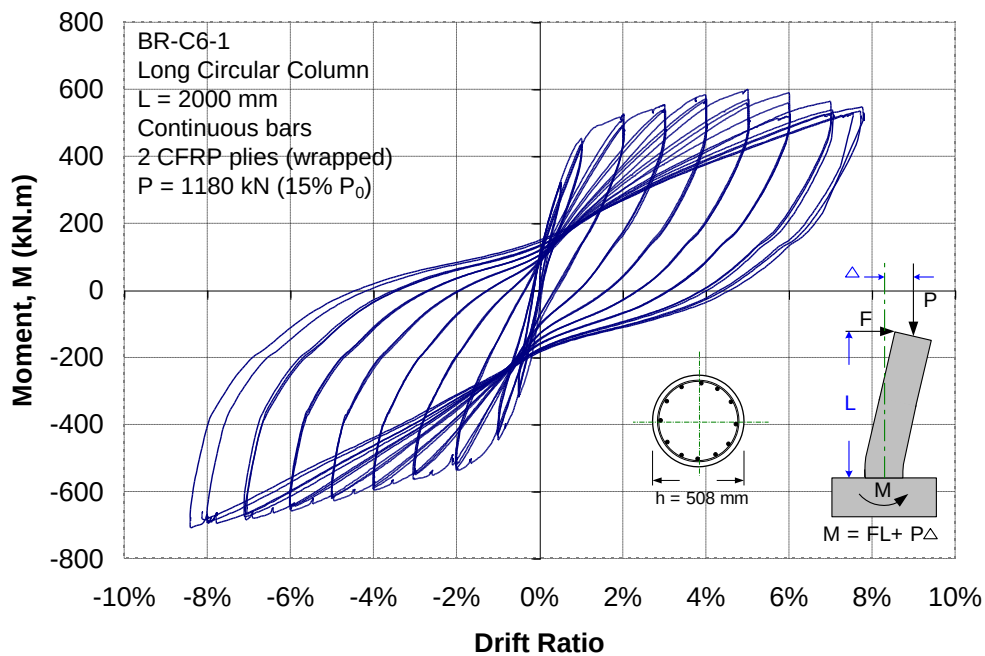
**Fig 4.9** Maximum fiber strains in columns BR-C10-1 & BR-C10-2



**Fig 4.10** Moment-drift envelopes for control column BR-C10 and retrofitted columns BR-C10-1 & BR-C10-2



a) Control column BR-C6 (Mes, 1999)



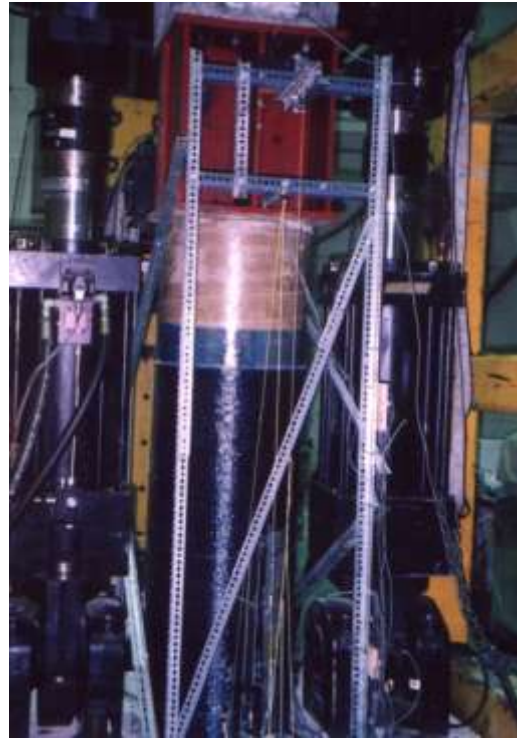
b) Retrofitted column BR-C6-1, 2 plies of CFRP

**Fig. 4.11** Moment-drift hysteretic relationships for control column BR-C6 and retrofitted column BR-C6-1



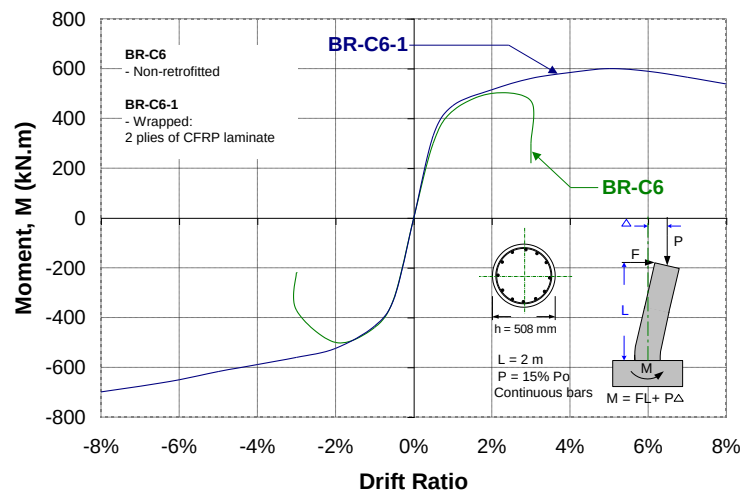


Control column BR-C6 after test



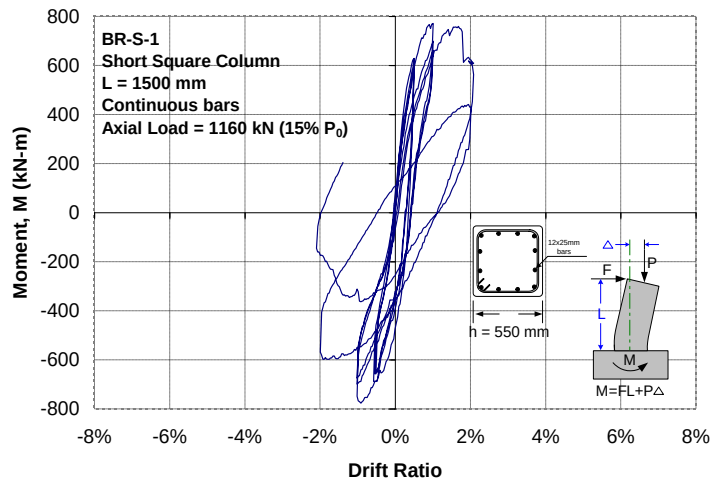
Retrofitted column BR-C6-1  
(2 plies of CFRP)

**Fig. 4.12** Long circular columns with continuous bars

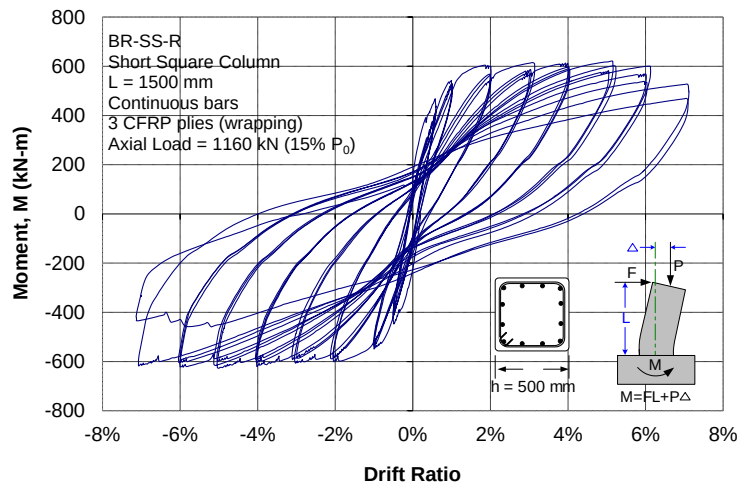


**Fig. 4.13** Moment-drift envelopes for control column BR-C6 and retrofitted column BR-C6-1



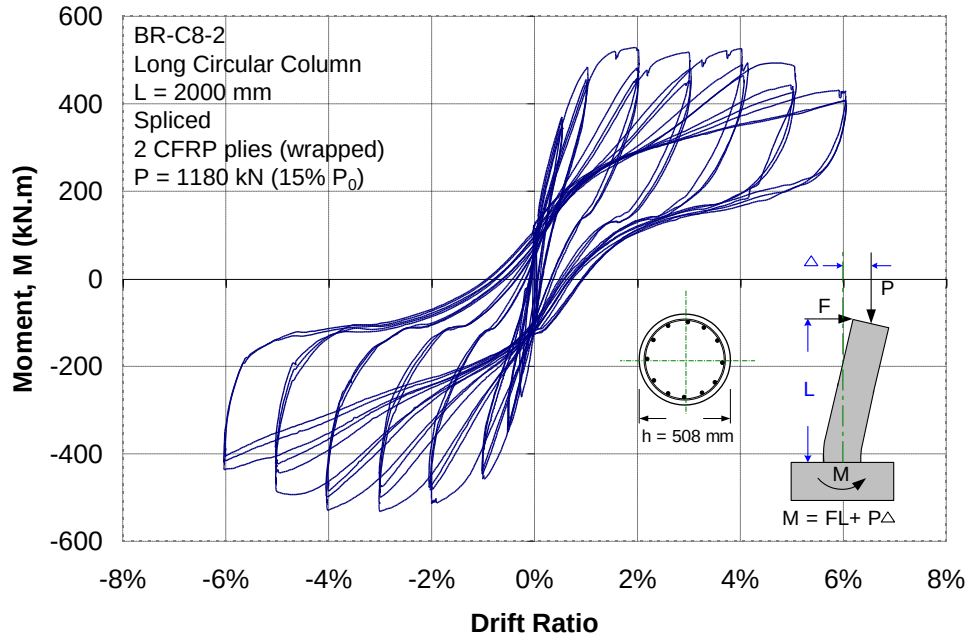


a) Control column BR-S1 (Yalcin, 1997)

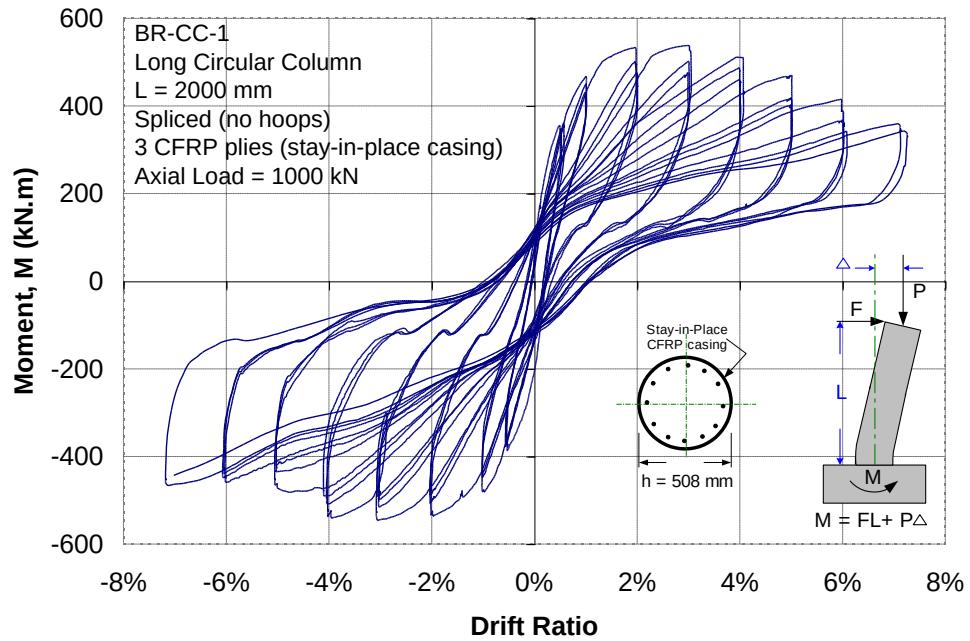


b) Retrofitted column BR-SS-R

**Fig. 4.14** Moment-Drift hysteretic relationships for control column BR-S1 and retrofitted column BR-SS-R

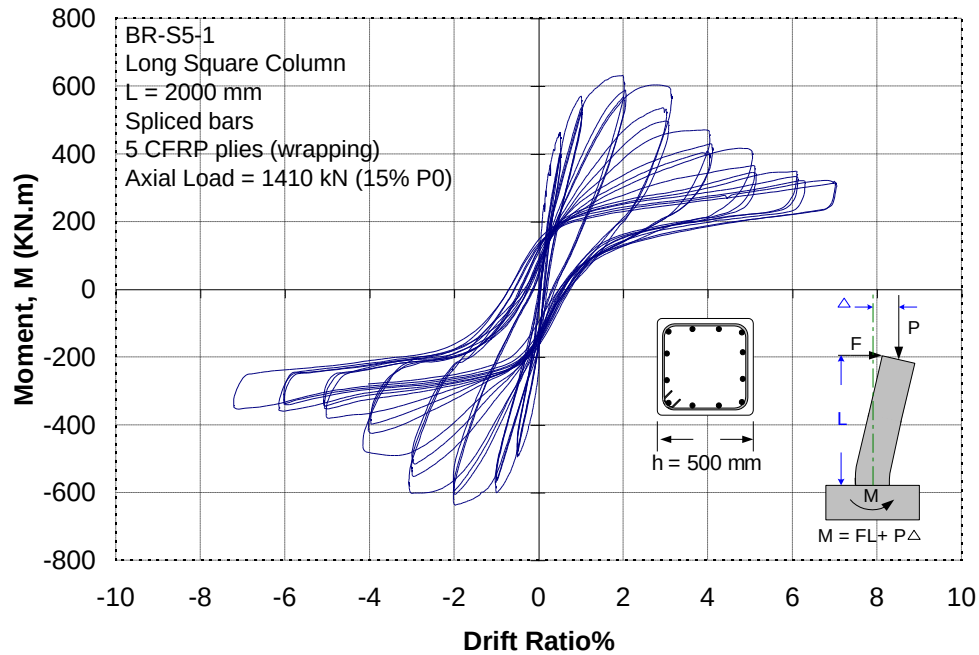


a) Retrofitted column BR-C8-2, 2 plies of CFRP

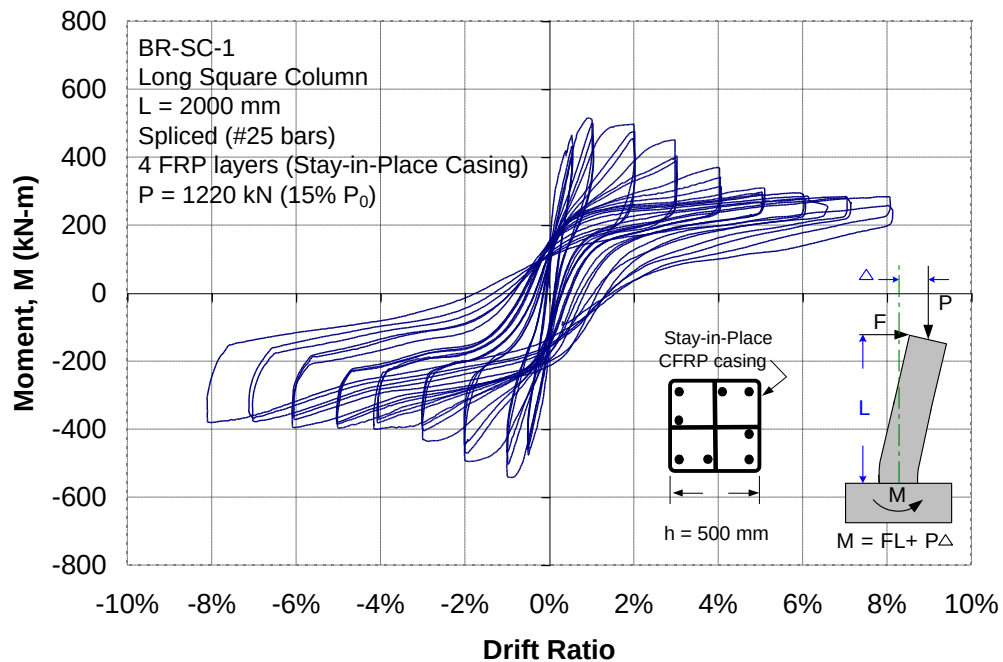


b) Column BR-CC-1, 3 plies, stay-in-place CFRP formwork

**Fig. 4.15** Moment-drift hysteretic relationships for retrofitted column BR-C8-2 and column BR-CC-1 in stay-in-place formwork

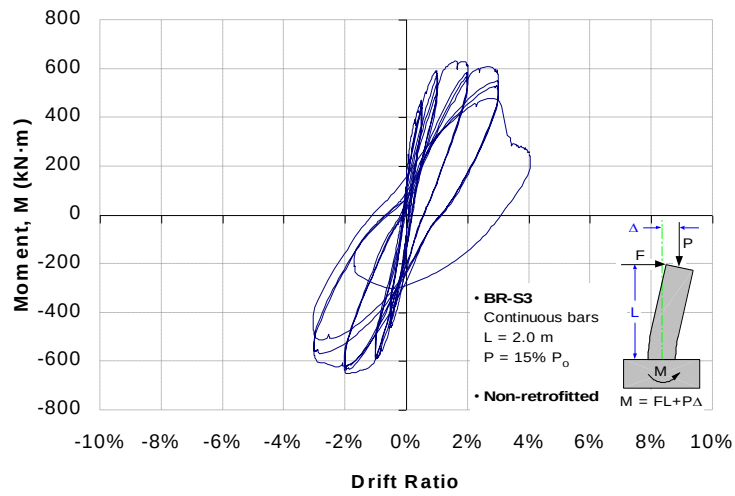


a) Retrofitted column BR-C5-1, 5 plies of CFRP, (Elnabelsy, 2000)



b) Column BR-SC-1 with 4 plies CFRP casing, CFRP crossties, and 25 mm reinforcement

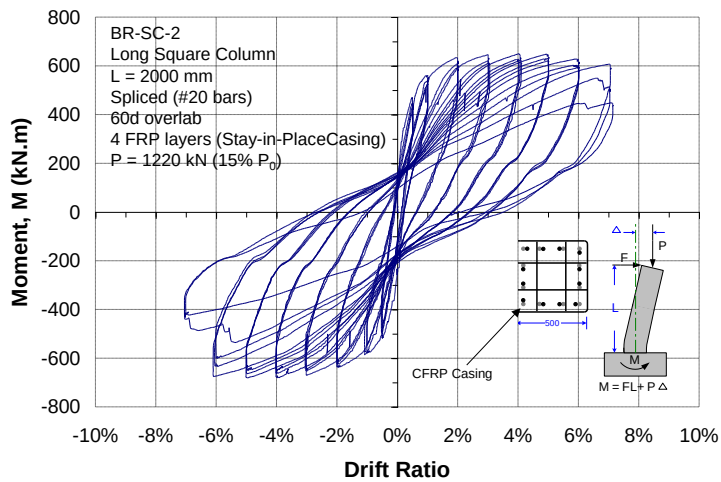
**Fig. 4.16** Moment-drift hysteretic relationships for wrapped column BR-C5-1 and column BR-SC-1 with stay-in-place CFRP formwork



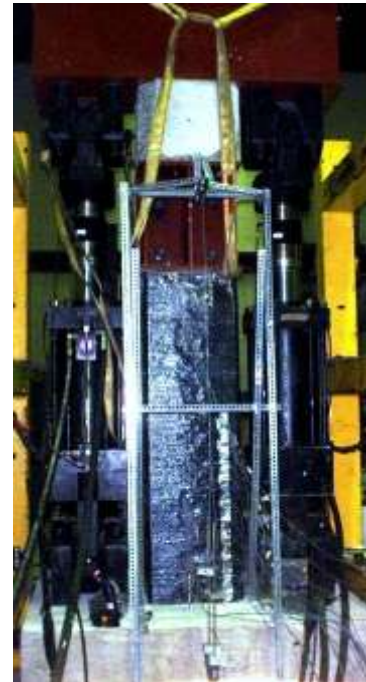
a) Column BR-S3 with continuous bars (Mes, 1999)



Column BR-S3 after test



b) Column BR-SC-2 with spliced bars (60  $d_b$  lap splice length)



Column BR-SC-2

**Fig. 4.17** Moment-drift hysteretic relationships for control column BR-S3 and retrofitted column BR-SC-2

# **CHAPTER 5**

## **COLUMN ANALYSES AND DEVELOPMENT OF DESIGN PROCEDURE**

### **5.1 GENERAL**

Analytical research was conducted on bridge columns externally wrapped with CFRP as well as columns cast using prefabricated CFRP casings to verify the applicability of existing analytical techniques and models in predicting the behaviour of these columns in flexure and shear. A design procedure was developed for concrete confinement and lap splice clamping. The applicability of the shear design procedure of CSA S806-2012 for FRP jacketing is also verified. Flexural analyses of columns were conducted by employing plane-section analysis to generate sectional response within the entire range of inelastic deformations. This necessitated the incorporation of existing material models, including a confined concrete model. A computer program (RC Section) was used to construct moment-curvature relationships, which were used to conduct member analysis to establish moment-lateral drift relationships. The effects of yield penetration into the column footing, which led to anchorage slippage, were also incorporated into the flexural analysis.

In order to conduct the sectional analysis, the concrete confinement model developed by Saatcioglu and Razvi (2002) was modified to incorporate the effects of CFRP as transverse confinement reinforcement. Furthermore, the experimentally recorded transverse strain limits were adopted to assess the effectiveness of CFRP as confinement reinforcement and to devise a design method for lap splice clamping.

The columns tested were further analyzed using the available analytical techniques for shear with due considerations given to diagonal tension capacity of cracked concrete and

resistance provided by external CFRP sheets. The analysis results provided the required verification of analytical techniques for use in practice.

The results of analytical research, including recommended design procedures are presented in this Chapter.

## **5.2 FLEXURAL ANALYSIS**

Flexural analysis of columns was conducted in two stages; i) column sectional analysis and ii) column member analysis.

### **5.2.1 Sectional Analysis for Flexure**

Standard plane-section analysis of reinforced concrete sections was conducted by using the assumption of plane sections before bending remaining plane after bending, enabling a linear relationship between sectional strains across the depth of a section. The contribution of tension concrete to sectional capacity was neglected. Material constitutive models were used to establish the relationships between strains and stresses for concrete and reinforcing steel. The moment-curvature relationships were then established by calculating the moment resistance corresponding to different strain profiles throughout the entire range of inelastic deformations as discussed later.

### **Confined Concrete Model**

An important aspect of the retrofit strategy was to improve the confinement of compression concrete to enhance inelastic deformability. This was true for both FRP wrapped columns and columns cast in FRP stay-in-place formwork. Therefore, it was essential to adopt an appropriate constitutive model for confined concrete. The model proposed by Saatcioglu and Razvi (1992) was used for this purpose. This model was developed to construct the stress-strain relationship of confined concrete on the basis of the concept of equivalent uniform confinement pressure. The model is applicable to normal strength concrete, though a modified version of the same model is also available for high-strength concrete (Razvi and Saatcioglu 1999). It incorporates the relevant parameters of confinement, such as the type, volumetric ratio, spacing, yield strength and arrangement of transverse reinforcement, as well as the cross-sectional geometry and

strength of concrete for internally reinforced concrete columns. The equivalency between individual ties and continuous coverage of column face by CFRP sheets is first established through experimental observations. The confined concrete strength and corresponding strain are expressed in terms of equivalent uniform confinement pressure provided by CFRP sheets. Fig. 5.1 shows the model, which consists of a parabolic ascending branch, followed by a linear descending segment. The model reduces to that proposed by Hognestad (1951) if the effect of confinement is negligible. The following expressions define the stress-strain relationship for ascending and descending branches, respectively.

$$f_c = f_{cc}' \left[ 2 \left( \frac{\varepsilon_c}{\varepsilon_1} \right) - \left( \frac{\varepsilon_c}{\varepsilon_1} \right)^2 \right]^{\frac{1}{1+2K}} \quad \varepsilon_c \leq \varepsilon_{01} \quad [5.1]$$

$$f_c = f_{co}' - (\varepsilon_c - \varepsilon_{01}) \left( \frac{0.15 f_{co}'}{\varepsilon_{085} - \varepsilon_{01}} \right) \quad \varepsilon_{01} < \varepsilon_c \leq \varepsilon_{cu} \quad [5.2]$$

where,  $f_c$  and  $\varepsilon_c$  are concrete stress and strain. The unconfined concrete strength in member and the corresponding strain are denoted as  $f_{co}'$  in MPa and  $\varepsilon_{01}$ . Strain at 85% of strength on the descending branch is denoted by  $\varepsilon_{085}$ .

Fig. 5.2 illustrates confinement pressures induced by different arrangements of column reinforcement. The confined concrete strength ( $f_{cc}'$ ) is computed using equivalent uniform confinement pressure in MPa ( $f_{le}$ ), which is less than the average pressure ( $f_l$ ) or the actual pressure peaks at corner or nodal points where transverse reinforcement legs are present. The confined concrete strength can be computed as the sum of unconfined strength in column and the incremental effect of equivalent uniform lateral pressure, as shown in Eq. 5.3. It was experimentally observed that the rate of increase in concrete strength due to lateral pressure decreases with the magnitude of pressure (Saatcioglu and Razvi 1982). This is introduced into Eq. 5.3 through coefficient  $k_1$ .

$$f_{cc}' = f_{co}' + k_1 f_{le} \quad [5.3]$$

where;

$$k_1 = 6.7(f_{le})^{-0.17} \quad [5.4]$$

$$f_{le} = k_2 f_l \quad [5.5]$$

$$f_l = \frac{\sum_{i=1}^q (A_{st} f_{st} \sin \alpha)_i}{sb_c} \quad [5.6]$$

$$K = \frac{k_1 f_{le}}{f_{co}} \quad [5.7]$$

$k_2$  is the coefficient that relates the average confinement pressure acting on column face to equivalent uniform pressure, reflecting the efficiency of confinement reinforcement. It assumes a smaller value for widely spaced reinforcement and can reach up to 1.0 when full efficiency of concrete confinement is attained as in the case of circular hoops/spirals and closely spaced and laterally supported longitudinal reinforcement. Eqs. 5.8 and 5.9 define the confinement efficiency coefficient  $k_2$ .

$$k_2 = 0.15 \sqrt{\left(\frac{b_c}{s}\right) \left(\frac{b_c}{s_l}\right)} \leq 1.0 \quad \text{For rectilinear reinforcement} \quad [5.8]$$

$$k_2 = 1 \quad \text{For closely spaced circular spirals} \quad [5.9]$$

The confinement efficiency coefficient  $k_2$  may be different for FRP jacketed square and rectangular columns, which provides continuous coverage of the column surface, when compared with columns with conventional reinforcement having only the perimeter ties. Ozbakkaloglu and Saatcioglu (2007) experimentally observed that internal column ties spaced at  $b_c/4$  are approximately 75% as effective as FRP jackets with continuous coverage of column side faces (or FRP jackets 1.33 times as effective as perimeter hoops spaced at  $b_c/4$ ). Furthermore, in columns with FRP jackets  $s_l = b_c$  since the FRP jacket provides lateral support to only the perimeter longitudinal bars. This results in  $k_c = 0.4$  for square and rectangular jackets. Coefficient  $k_c$  remains as 1.0 for circular jackets where hoop tension provides uniform lateral confinement pressures, whether it is closely spaced hoops, spirals or FRP jackets.



The descending branch of the proposed model for confined concrete consists of a linear segment originating from the peak concrete stress and the corresponding strain ( $\varepsilon_1$ ), sloping down to a strain ( $\varepsilon_{85}$ ) corresponding to 85% of the peak stress (Eqs. 5.10 and 5.11). This linear segment continues with a decaying slope down to a strain level corresponding to 20% of the peak stress. The residual strength is then kept constant at 20% of peak strength, simulating an average contribution of damaged concrete to strength (Fig. 5.1).

$$\varepsilon_1 = \varepsilon_{01}(1 + 5k_3k) \quad [5.10]$$

$$\varepsilon_{85} = 260k_3\rho_c\varepsilon_1(1 + 0.5k_2(k_4 - 1)) + \varepsilon_{085} \quad [5.11]$$

### Model for Longitudinal Reinforcement

The longitudinal column reinforcement was modeled with a bi-linear stress-strain relationship, followed by a strain hardening region. The strain hardening segment consisted of four linear segments, idealizing the curved portion of stress-strain relationship between the onset of strain hardening and the steel rupturing in tension. This was done by idealizing the experimentally obtained stress-strain relationship between 1.0% strain at the onset of strain hardening and 13% strain at steel rupturing. Fig. 5.3 illustrates the analytical model used for longitudinal reinforcement. It should be noted that the stress-strain relationship in compression was assumed to be the same as that in tension.

#### 5.2.2 Member Analysis for Flexure

Once the sectional response was established in the form of moment-curvature relationship, flexural deformations were then computed over the length of columns by constructing the distribution of curvatures over the column height. Incrementally increasing and linearly varying moments between the column tip and the critical column section at the footing interface enabled the construction of curvature distributions at selected stages of loading. Integration of curvatures resulted in column rotations and the moments of the area under the curvature diagrams relative to the tip of columns provided

horizontal column displacements due to flexure. The ratio of horizontal column displacement and column height resulted in a drift ratio. Drift ratios at selected stages of loading were computed. These points corresponded to; i) cracking moment, ii) first yielding of extreme tension reinforcement, iii) onset of strain hardening of extreme tension reinforcement, iv) peak moment resistance, v) 80% of peak moment resistance beyond the peak (at 20% strength decay), and vi) a point between the peak and the 80% strength decay corresponding to the average of curvatures at these two moment resistances. For each point, the moment and corresponding curvature was obtained from the analytical moment-curvature relationship, established as described in Sec. 5.2.1.

Beyond the peak moment resistance, plastic hinge starts forming as the member starts experiencing plastic deformations and decay under increasing curvatures.

### **Plastic Hinge Region**

The plastic hinge length is usually idealized by assuming a region of constant curvature. A number of empirical expressions were suggested by previous researchers for this constant curvature segment (plastic hinge length) at ultimate (Corley “1966”, Mattock “1964”, Sawyer “1964”). The expression suggested by Sawyer, given in Eq. 5.12, was used for the computation of ultimate deformation at 20% strength decay beyond peak resistance (point “v” above). Fig. 5.4 illustrates the distribution of curvatures at each of the control points used to compute lateral drift.

$$l_p = 0.25d + 0.075Z \quad [5.12]$$

$l_p$  is the plastic hinge length ,

$d$  is the effective depth of section, measured to the centroid of extreme layer of tension reinforcement, and

$Z$  is the distance between the critical section and the point of inflection (shear span).

### **5.3 ROTATION AND LATERAL DRIFT DUE TO ANCHORAGE SLIP**

Reinforced concrete members, subjected to significant flexural inelasticity at member ends experience yield penetration into the adjoining members. Yielding of anchored longitudinal reinforcement within the adjacent member results in the elongation and possible slippage of reinforcement outside the element (within the adjacent member). This results in significant softening of anchorage, generating additional deformations at member ends which are not accounted for in flexural analysis. This phenomenon, known as “anchorage slip” can be as significant as those caused by flexure, especially if the reinforcement enters into the strain hardening range of deformations. Tests on concrete columns show that anchorage slip plays an important role on total rotation of columns under lateral loading. In the case of bridge columns where the axial compression is low (10% to 20% of the column concentric capacity), anchorage slip has a larger effect than building columns under heavier axial compressive loads. Anchorage slip generates permanent rigid body rotations at member ends, contributing significantly to lateral drift. In this research program, lateral drift caused by anchorage slip was computed based on the procedure suggested by Alsiwat and Saatcioglu (1992). Accordingly, the embedded bar experiences two components of inelastic deformation; i) due to the extension of anchored reinforcement and ii) the slippage of the bar as a whole, prior to anchorage failure. Well anchored bar, with sufficient embedment length do not experience slippage. Often, the slippage is prevented and the anchorage is improved by using hooks.

#### **5.3.1 Extension of Reinforcement**

Extension of reinforcement near the end of a member results in an accumulated elongation of reinforcement at the interface of two adjacent members. Therefore, the extension of column reinforcement within its footing results in an elongation of column longitudinal reinforcement which generates a wide crack at the column-footing interface. According to Alsiwat and Saatcioglu (1992), this elongation can be computed by constructing the strain distribution along the embedment length of reinforcement and calculating the area under the strain diagram. This area gives the extension (elongation)

of reinforcing bar within the adjoining element. **Fig. 5.5** illustrates a typical strain distribution of an embedded re-bar with elastic and inelastic segments. The inelastic segment consists of yield plateau and strain hardening regions. If the reinforcement is embedded in an element with unreinforced cover concrete, the end segment pulls-out with a cone of concrete. This region is referred to as the “pull-out cone,” with strains in steel remaining constant. The length of each segment of the embedded re-bar can be computed using bond stress between steel and concrete, and the principles of equilibrium. Once the strain distribution and length of each segment are established, the strain profile can then be integrated to find the extension of reinforcement. The computation of bond stresses and segmental lengths are explained in the following paragraphs.

### Elastic Region

Re-bars are strained elastically under increasing tension, until the strained segment approaches their development length. Though the estimation of bond stress is a challenging task, it is generally acceptable to employ an average empirical value for elastic bond, as established by tests. The elastic bond, as recommended by ACI Committee 408, has been adopted in the model used in the current investigation. The expressions for elastic bond ( $u_e$ ) and the elastic bar length are presented below.

$$u_e = u_{ACI} = \frac{f_y d_b}{4l_d} \quad [5.13]$$

Where,

$$l_d = \frac{440A_b}{K' \sqrt{f_c}} \frac{f_y}{400} \geq 300mm, \quad K' = 3d_b \quad [5.14]$$

$f_y$  is the nominal yield strength of steel reinforcement in *MPa*,

$d_b$  is the bar diameter in mm,

$l_d$  is the development length in mm, and

$f_c$  is the concrete compression strength in *MPa*.

Once the bond stress is calculated,  $l_e$  the elastic length can be derived from;

$$l_e = \frac{f_s d_b}{4u_e} \quad [5.15]$$

$f_s$  is the maximum stress in steel and it reaches  $f_y$  when the elastic bar length reaches the development length.

### **Yield Plateau Region**

As the tension force in steel increases, the bar starts yielding. Further straining of the bar results in the penetration of yielding. If the yield plateau has a positive (non-zero) slope, the length of this region also becomes non-zero. Increased post-yield strains result in the development of frictional bond ( $u_f$ ) between concrete and steel. The value of average constant frictional bond stress can be determined experimentally. Eq. 5.16 shows the expression suggested by Pochanart and Harmon (1989):

$$u_f = \left( 5.5 - 0.07 \frac{S_L}{H_L} \right) \sqrt{\frac{f_c'}{27.6}} \quad [5.16]$$

$S_L$  is the clear spacing between the lugs (deformations) of the bar and  $H_L$  is the height of these lugs above the reinforcement surface. By knowing the frictional bond stress ( $u_f$ ), the yield plateau length can be computed as;

$$L_{yp} = \frac{\Delta f_s d_b}{4u_f} \quad [5.17]$$

where,  $\Delta f_s$  is the difference in steel stress between the start and end points of yield plateau region.

### **Strain Hardening Region**

Further straining of reinforcement, under increasing inelasticity of the attached member, results in the strain hardening of steel which begins at approximately 1% strain. Strain hardening provides additional reserved capacity, which may be as high as 125% of yield strength. The bond stress in this region can be assumed to be equal to the frictional bond. The requirement of force equilibrium and the assumption of constant frictional bond

within the segment can be used to establish the length of this region, as illustrated in Eq. 5.18.

$$L_{sh} = \frac{\Delta f_s d_b}{4u_f} \quad [5.18]$$

### **Pullout Cone Region**

Pullout cone occurs when concrete (usually the cover) breaks loose at the onset of yielding. The length of the cone is usually equal to the cover concrete. The pullout cone does not occur if this region is reinforced. When cover is small, the effect of the pull-out cone may be negligible. In this research project, the effect of the pull-out cone was neglected

### **Total Extension of Reinforcement:**

Bar extension can be computed through the integration of strains along the embedment length (Fig. 5.5) and can be determined from the following equation:

$$\delta_{ext} = \varepsilon_s L_{pc} + 0.5(\varepsilon_s + \varepsilon_{sh})L_{sh} + 0.5(\varepsilon_{sh} + \varepsilon_y)L_{yp} + 0.5\varepsilon_y L_e \quad [5.19]$$

The member-end rotation associated with the extension of reinforcement in the adjoining member can be obtained from

$$\theta_e = \frac{\delta_{ext}}{d - c} \quad [5.20]$$

Where,  $d$  and  $c$  are the depths of the section measured from the extreme compression fiber to the centroid of the tension steel and the neutral axis, respectively. The effect of anchorage slip rotation is to induce a linearly increasing displacement profile. The horizontal deflection at the tip of the column can then be computed from the following equation.

$$\Delta_{horz} = L(\theta_e) \quad [5.25]$$

Where,  $L$  is the column shear span.

### **5.3.2 Comparisons of Analytically Generated Moment-Drift Envelopes with Experimentally Recorded Hysteretic Relationships**

The analytical procedure explained in the preceding section was used to construct the envelopes of moment-lateral drift relationships for the columns tested. The lateral drifts included inelastic flexural deformations, as well as those caused by anchorage slip. The experimental data were compared with the analytical results to evaluate if the predicted behavior and strength of the columns were close to the actual values. An important short coming of the flexural analysis utilized was the lack of a model that could incorporate the mechanism of splice slippage and the resulting strength decay. Therefore, significant discrepancies were observed for spliced columns between the analytical and experimental relationships, though the strength agreement at approximately the yield point of longitudinal reinforcement, just before 1% drift, was reasonably good. Once the reinforcement started slipping within the splice region, the strength during load reversal showed an increased decay. The discrepancy in results became wider as the longitudinal reinforcement continued slipping in the splice region. Shear deformations were believed to be small relative to those caused by flexure and negligible in most cases. Hence they were not included in the comparisons.

Comparisons of analytical and experimental Moment-Drift relationships are presented below.

#### **5.3.2.1 Columns wrapped with CFRP sheets**

Figs. 5.6 through 5.9 depict columns wrapped with CFRP sheets. Column BR-C8-2, with spliced reinforcement, has a long shear span with increased shear stress reversals. The strength envelope computed analytically provided good agreement with the experimentally recorded hysteretic relationship up to 2% drift although the moment capacity was over estimated at 5% drift. Beyond 2% drift, bar slippage took place within the splice region because of the short splice length used in the column ( $20d_b$ ), as evidenced by the pinching of hysteresis loops. This was accompanied by strength degradation. As the bar slippage was not modelled in this analysis, the analytical envelop curve showed higher moment resistance relative to experimentally recorded deformations

under increasing lateral drift. The analytical envelope was generated up to 7% drift (0.035 extreme fibre strain in concrete) based on the assumption that there was no slippage of the main longitudinal reinforcement within the splice region in the hinge region. This specimen sustained 5% drift ratio, compared to the control specimen BR-C8 (Bausijour, 2000), which failed at 2%. The retrofitting procedure improved the shear capacity by providing additional transverse reinforcement.

Column BR-C6-1 has a long shear span and continuous bars. The envelope computed analytically provided a very good agreement with the experimentally recorded hysteretic relationship for both strength and inelastic deformability through the entire deformation range, verifying the applicability of the analysis procedure employed, including the analytical models adopted. The buckling of longitudinal reinforcement at high inelastic drift levels (beyond 6% drift) was not modelled in the stress-strain relationship of reinforcement in compression. This may explain the slight overestimation of the analytical envelop curve on one side, at 7% and 8% drift level. The analysis technique was able to capture the improved ductility generated by FRP jacketing, due to the incorporation of the concrete confinement model. The companion columns (BR-C6), without the retrofit failed at 2% lateral drift (Mes, 1999).

Column BR-C10-2 has a short shear span and deficient lap splice of longitudinal reinforcement in the plastic hinge region. The analytical moment-drift envelop curve is similar to that generated earlier for column BR-C8-2, which has similar characteristics, except for the shear span. Once again, the computed envelope agreed with experimental curves up to 1% drift and started showing overestimation of strength at higher drift levels. This is attributed to the slippage of longitudinal reinforcement as evidenced by the pinching of hysteresis loops and associated strength decay. With slipping bars, the column was able to maintain its drift capacity with little strength decay up to about 4% drift, compared with the control column without the CFRP retrofit (BR-C10) which failed at 2% drift (Bausijour, 2000).



The 4<sup>th</sup> column that was wrapped with CFRP is BR-SS-R. The column has a square cross section and continuous reinforcement. The analytical and experimental moment-drift relationships are compared in Fig. 5.9. The computed response compares well with experimentally recorded moment-drift curves, verifying the applicability of the analysis procedure employed. However, this column has high shear stress reversals because of its short shear span. The CFRP jacket provided sufficient shear reinforcement to suppress shear failure and change the behaviour to flexure, allowing the yielding of the longitudinal reinforcement and dissipation of energy through the formation of plastic hinge. Nevertheless, the high shear stress reversal are believed to cause the deterioration of concrete in higher levels of drifts, explaining the slight difference between the analytical envelop curve and the strength levels indicated by the hysteretic relationships beyond the 3% drift cycles. The companion column, without the FRP jacket failed at 1% drift in shear, as previously discussed (Yalcin, 1997).

#### 5.3.2.2 Columns cast in CFRP Stay-in-Place formwork

Analytical and experimental moment-drift envelopes for columns cast in stay-in-place formwork are depicted in Figs. 5.10 through 5.12. Column BR-CC-1 in a circular CFRP casing is comparable to CFRP wrapped BR-C8-2, except that the former has no steel ties. The computed envelope shows agreement with experimentally recorded hysteretic relationship up to 3% drift, after which noticeable strength decay took place due to the slippage of spliced reinforcement as indicated by the pinching of hysteresis loops. This resulted in the deviation of analytical curves from the experimental relationship beyond the 4% drift level.

Moment-drift envelope for BR-SC-1 with square a casing and single CFRP crossties is illustrated in Fig 5.11. This column was modeled with the presence of internal crossties reflected in the concrete confinement model, as it affects the distribution of confinement pressure ( $k_2$  parameter). The correlation between the analytically computed envelop curve and the experimental hysteretic relationship is good until the onset of bar slippage within the plastic hinge region starting 2% drift. The effect of lap splice deficiency is more pronounced in columns with square CFRP casings than those with circular tubes

used as stay-in-place formwork. Otherwise the strength of columns with CFRP stay-in-place formwork is computed accurately by the analytical procedure.

Column BR-SC-2 with a square stay-in-place formwork, incorporating double CFRP crossties had sufficient longitudinal splice length ( $60d_b$ ) and performed well as if the bars were continuous. The use of double crossties contributed to the efficiency of confinement, promoting ductile response. Fig. 5.12 shows the comparison of analytically computed envelop with experimentally recorded moment-drift hysteretic relationship. The correlation of the two sets of results is excellent, confirming that the analytical approach can be used to estimate force and deformation characteristics of columns in CFRP stay-in-place formwork with internal crossties.

## 5.4 SHEAR STRENGTH ANALYSIS

The columns tested were analyzed using CSA S806-2012 Simplified Method to verify the applicability of this existing technique for the computation of column shear capacity when the columns were externally wrapped or incased with CFRP sheets in the transverse direction.

### 5.4.1 CSA S806-2012 Shear Strength Enhancement

The method presented in CSA S806-2012 is adopted to compute shear capacity of columns. The shear resistance of a column strengthened by FRP laminates with fibers oriented at an angle  $\theta \geq 75^\circ$  to the longitudinal axis of the column is computed as the summation of contributions from concrete, steel and FRP, as follows:

$$V_r = V_c + V_s + V_F \leq V_c + 0.6\lambda\phi_c\sqrt{f'_c}A_{cv} \quad [5.26]$$

Where;

$$V_c = 0.2\phi_c\lambda\sqrt{f'_c}A_{cv} \quad [5.27]$$

The transverse reinforcement is perpendicular to the longitudinal axis of the member, and it can be calculated from:

$$V_s = \frac{\phi_s A_v f_y d}{s} \quad [5.28]$$

The contribution of the laminate  $V_F$  is calculated from:

$$V_F = 2\phi_F t_F f_F D \quad [5.29]$$

Where;  $f_F = 0.006E_F \leq f_{Fu}$  [5.30]

The above expressions provide shear resistance for design purposes, and hence incorporate material resistance factors  $\phi_s = 0.85$ ,  $\phi_c = 0.65$  and  $\phi_F = 0.75$  along with the mass density factor  $\lambda$ . For the nominal shear capacities and for normal density concrete, these factors should be taken as 1.0. Although the columns tested in the current investigation were subjected to axial compression, Eq. 5.28 does not account for the contribution of axial compression to concrete shear resistance.

Table 5.1 includes experimentally recorded maximum applied shear values along with computed values. Both factor shear resistances and nominal shear capacities are listed in the table. Computed nominal shear capacities are plotted on the hysteretic relationships obtained from column tests. These comparisons are illustrated in Figs 5.13 to 5.19. Since all the columns failed in flexure, the corresponding applied shear forces are lower than the nominal shear capacities.

**Table 5.1** Experimentally recorded maximum applied shear values along with computed values

| <i>Column</i>   | <i>V<sub>c</sub> (kN)</i> | <i>V<sub>s</sub> (kN)</i> | <i>V<sub>F</sub> (kN)</i> | <i>V<sub>r</sub> (kN)</i> | <i>V<sub>n</sub> (kN)</i> | <i>V<sub>exp</sub> (kN)</i> | <i>V<sub>n</sub> / V<sub>exp</sub></i> |
|-----------------|---------------------------|---------------------------|---------------------------|---------------------------|---------------------------|-----------------------------|----------------------------------------|
| <b>BR-C8-2</b>  | 102                       | 58                        | 292                       | 452                       | 635                       | 250                         | 2.5                                    |
| <b>BR-C10-2</b> | 147                       | 69                        | 351                       | 567                       | 760                       | 450                         | 1.7                                    |
| <b>BR-C6-1</b>  | 102                       | 58                        | 292                       | 452                       | 610                       | 250                         | 2.4                                    |
| <b>BR-CC-1</b>  | 95                        | 0                         | 439                       | 534                       | 720                       | 260                         | 2.75                                   |
| <b>BR-SC-1</b>  | 117                       | 0                         | 405                       | 522                       | 700                       | 250                         | 2.8                                    |
| <b>BR-SC-2</b>  | 119                       | 0                         | 576                       | 695                       | 926                       | 300                         | 3.0                                    |
| <b>BR-SS-R</b>  | 117                       | 58                        | 432                       | 607                       | 810                       | 390                         | 2.0                                    |

## 5.5 PROPOSED DESIGN APPROACH FOR EXTERNAL FRP COMPOSITES

### 5.5.1 DESIGN FOR CONCRETE CONFINEMENT

A displacement based design procedure was developed for column confinement by Saatcioglu and Razvi (2002) for conventional steel reinforced concrete columns with lateral drift as the performance criterion. The design approach was based on drift capacities of columns under different levels of confinement and axial compression. The drift capacity was computed through column analysis under monotonically increasing lateral loading (push-over analysis). The analysis technique incorporated analytical models for concrete confinement (Saatcioglu and Razvi 1992, Razvi and Saatcioglu 1999a), steel strain hardening and buckling (Yalcin and Saatcioglu 2000), the formation and progression of plastic hinging (Razvi and Saatcioglu 1999b), and anchorage slip simulating the progression of yielding into the adjoining member and the resulting extension of reinforcement (Alsiwat and Saatcioglu 1992). The analysis procedure and the analytical models had been verified extensively against experimental data. A large number of column analyses were conducted to establish the significance of design parameters on lateral drift. The results indicated that the amount, grade, spacing and arrangement of confinement reinforcement, as well as the level of axial compression, concrete strength, cover-to-core area ratio and shear-span-to-depth ratio played important roles on the drift capacity. The percentage of longitudinal reinforcement also played a role, though to a lesser extent. The analysis results indicated that it was possible to combine the majority of these design parameters into a single ratio  $r$ , expressed in Eq. 5.31, indicating that columns with the same  $r$  ratio would develop approximately similar drift capacities irrespective of the variations in individual parameters that make up the ratio (Saatcioglu and Razvi 2002).

$$r = \frac{\rho_c f_{yh}}{f'_c \left[ \frac{A_g}{A_c} - 1 \right]} \quad [5.31]$$

The analytical research further indicated that the lateral drift capacity  $\delta$  improved with increasing  $r$  and decreased with increasing axial compression  $P$  (expressed as a fraction of column concentric capacity  $P_0$ ). Furthermore, the efficiency of transverse reinforcement arrangement as expressed by coefficient  $k_c$  played an important role on column drift capacity (Saatcioglu and Razvi 2002). It was suggested that, Eq. 5.32 would provide a good estimate of the relationship between lateral drift capacity  $\delta$  and the parameter  $r$  (Saatcioglu and Razvi 2002).

Where:

$$r = 14 \frac{1}{\sqrt{k_c}} \frac{P}{P_0} \delta \quad [5.32]$$

$$k_c = 0.15 \sqrt{\frac{b_c}{s} \frac{b_c}{s_t}} \leq 1.0 \quad [5.33]$$

$$\frac{P}{P_0} \geq 0.2 \quad [5.34]$$

The confinement efficiency coefficient  $k_c$  in Eq. 5.33 was found to be 1.0 for circular spirals and circular hoops. It varied with the spacing of transverse reinforcement “ $s$ ” and the spacing of laterally supported longitudinal reinforcement “ $s_t$ ” for columns having rectilinear reinforcement, so long as the tie spacing was not greater than half the column cross-sectional dimension ( $s \leq h/2$ ).

Eq. 5.32 was generated for columns with a shear-span-to-depth ratio  $L/h$  of 2.5. This level is near the lower end of  $L/h$  ratios used in practice and provides a conservative estimate of drift capacities for columns with higher  $L/h$  ratios. Eq. 5.32 was developed for longitudinal reinforcement ratio of 2%. The effects of longitudinal reinforcement and

shear-span-to-depth ratio on column deformability were found to be small and hence they were not introduced as separate parameters.

Equating Eqs. 5.31 and 5.32, and solving for reinforcement ratio  $\rho_c$  provides a design expression for the required amount of conventional confinement reinforcement in each cross-sectional direction.

$$\rho_c = 14 \frac{f'_c}{f_{yh}} \left[ \frac{A_g}{A_c} - 1 \right] \frac{1}{\sqrt{k_c}} \frac{P}{P_o} \delta \quad [5.35]$$

Eq. 5.35 can be adapted to columns externally wrapped with CFRP sheets or cast in CFRP stay-in-place formwork. One of the main differences between this and conventional steel reinforced column is the fact that the former does not have a cover concrete and the entire gross cross-sectional area of concrete is confined as core concrete. Therefore,  $(A_g/A_c - 1)$  approaches zero and hence should be replaced with the minimum value for which the above expression was derived, i.e., 0.3. Eq. 5.35 can then be written in terms of the CFRP thickness, relevant material strengths and factored design axial force quantities, as shown below:

$$t_j = 2D \frac{f'_c}{\phi_j f_{Fj}} \frac{\delta}{\sqrt{k_c}} \frac{P}{P_o} \quad [5.36]$$

Coefficient  $k_c$  represents the efficiency of confinement reinforcement. In circular columns, where confinement is provided by hoop tension,  $k_c = 1.0$  and the confinement efficiency is the highest. This confinement reinforcement develops uniform tension, which results in uniform (or near uniform) confinement pressure on column concrete. FRP jackets and FRP tubes around circular columns behave in a similar manner and develop hoop tension, resulting in uniform confinement pressure. Therefore,  $k_c = 1.0$  for circular FRP jackets and circular tubes. However, square and rectangular jackets and stay-in-place formworks behave similar to columns confined by perimeter hoops, with or

without internal crossties. The confinement efficiency coefficient  $k_c$  for these cases can be adopted from Eq. 5.33. Ozbakkaloglu and Saatcioglu (2007) experimentally observed that internal column ties spaced at  $b_c/4$  are approximately 75% as effective as FRP jackets or tubes with continuous coverage of column side faces (or FRP jackets 1.33 times as effective as perimeter hoops spaced at  $b_c/4$ ). Substituting  $s = b_c/4$  in Eq. 5.33 and increasing the coefficient 0.15 in the same equation by 1.33, the following expression can be obtained for confinement efficiency coefficient  $k_c$ .

$$k_c = 0.4 \sqrt{\frac{b_c}{s_t}} \leq 1.0 \quad [5.37]$$

Where,  $s_t$  is the spacing of internal FRP crossties in stay-in-place formwork. Columns with square FRP jackets or FRP casings without the internal FRP ties have  $s_t = b_c$ . This results in  $k_c = 0.4$  for square sections without the internal crossties. For rectangular jackets or tubes, with different  $b_c/s_t$  ratios in the two cross-sectional directions, an average value of  $k_c$  computed in each direction may be used. The observation of column performance with CFRP tubes and internal CFRP crossties indicates that the amount of fibres in each crosstie should be at least equal to the total amount of fibres within crosstie spacing ( $s$ ) in the FRP tube in the direction of crossties, divided by the number of crossties in the same direction.

Eq. 5.36 includes a capacity reduction factor ( $\phi_j$ ) which reflects potential imperfections in the FRP jacket and the quality of workmanship during the manufacturing process, especially when hand lay-up method is employed. A value of  $\phi_j \leq 0.8$  may be appropriate for use in design. Furthermore, the bridge columns tested under  $15\%P_o$  indicate that Eq. 5.36 does produce good correlations with test data at axial load levels as low as  $15\%P_o$ , as demonstrated in the next section, which is slightly below the lower limit of  $20\%P_o$  specified in Eq. 5.34 for transverse steel reinforcement. Hence, Eq. 5.38 may be used as the lower limit when Eq. 5.36 is employed for FRP jacket or stay-in-place formwork design.

$$\frac{P}{P_0} \geq 0.15 \quad [5.38]$$

An important parameter in Eq. 5.36 is  $f_{Fj}$ , which represents the transverse stress in FRP at the limit of confinement effectiveness. Unlike the steel confinement reinforcement, FRP composites do not have a yield point and yield plateau at which the stress becomes constant, i.e.,  $f_{yh}$ . The level of transverse stress in FRP transverse reinforcement varies considerably with factors that contribute to the lateral expansion of concrete in compression and the rigidity of FRP composite. Bridge columns tested under low levels of axial compression that are significantly below the balanced axial load fail by the rupturing of longitudinal reinforcement long before mobilizing the full capacity of FRP as discussed in Chapter 4. The FRP jacket in these columns is effective up to approximately an average strain value of 0.3% and 0.5% within the potential plastic hinge region of square and circular columns, respectively when the column axial load is at 15% of concentric compression capacity ( $P_0$ ). This is illustrated in Fig. 5.20, and these values are used in computing  $f_{Fj}$ .

$$f_{Fj} = 0.003 E_F \quad \text{For square columns} \quad [5.39]$$

$$f_{Fj} = 0.005 E_F \quad \text{For circular columns} \quad [5.40]$$

The design drift ratio  $\delta$  is established by the design engineer for expected level of column deformability. Member yielding (as opposed to sectional yielding) in flexure-dominant columns occurs at approximately 1% lateral drift ratio. Therefore, the ductility related force modification factor  $R_d$  specified in most seismic design codes can be used as drift demand ratio. In the National Building Code of Canada (2010), the ductility related force modification factor  $R_d = 4$  for fully ductile reinforced concrete frames, and  $R_d = 2.5$  for moderately ductile moment-resisting frames. These factors may be used as approximate drift demands for column retrofit and stay-in-place formwork design, corresponding to  $\delta = 4\%$  and  $\delta = 2.5\%$  drift ratios for full and moderate ductility, respectively. However, the selection of the design drift levels should be left with the engineer in charge, and the use of  $R_d$  factors may be regarded as minimum values, since providing extra drift capacity



can be justified for seismic retrofitting. This is especially true for columns in regions of moderate seismicity where minimum thickness of FRP jacket or FRP tube may provide higher drift capacities than implied in the above minimum design drift demands.

### 5.5.2 DESIGN FOR SPLICE DEFICIENCY

The experimental research presented in Chapters 3 and 4 indicated that the design procedure described in the preceding section for column confinement can also be used to improve the performance of splice regions in splice-deficient columns. The primary difference in this case is the limitation that needs to be imposed on the design transverse strain limit, and the effectiveness of the FRP. It was observed experimentally that beyond an average transverse strain of 0.0015 to 0.0020, the bond between the spliced reinforcement and the surrounding concrete cannot be maintained. Beyond this strain range, columns develop excessive pinching of hysteresis loops, indicating significant slippage of spliced reinforcement. This observation is based on the columns designed and built in the pre-1970's era, when the splice length was equal to  $20 d_b$ . It was further observed that this behavior can be prevented in circular columns, where the confinement pressure is uniform if the transverse strain is kept below 0.0015. Therefore, for retrofitting these columns it is recommended to use Eq. 3.36 with  $f_{Fj}$  limited to  $0.0015E_F$ . The same improvement could not be attained in square columns, though the use of CFRP jackets did improve the splice behavior somewhat. Until further research data become available on the relationship between the splice length and the characteristics of FRP jackets, it is recommended that FRP jacketing should *not* be used as clamping reinforcement in square and rectangular columns for improving the behavior of splices in potential plastic hinge regions.

$$t_j = 2D \frac{f'_c}{\phi_j f_{Fj}} \frac{\delta}{\sqrt{k_c}} \frac{P}{P_o} \quad [5.41]$$

Where  $f_{Fj} = 0.0015E_F$

Equation 5.41 applies to circular columns.

### 5.5.3 DESIGN FOR SHEAR

The design of FRP jackets and FRP stay-in-place formworks for shear can be done by following the conventional 45-degree truss analogy, adopted in CSA S806-2012. Accordingly, the shear resistance consists of contributions from concrete, internal transverse shear reinforcement and external FRP composite, as indicated in Eqs. 5.26 through 5.30. Column design using this approach did prevent brittle shear failure associated with diagonal tension stresses in the shear-dominant columns tested as part of the current investigation. The comparison of experimental shear force resistances with computed nominal capacities using the above approach is discussed in Sec. 5.4.

### 5.5.4 VERIFICATION OF THE PROPOSED DESIGN PROCEDURE

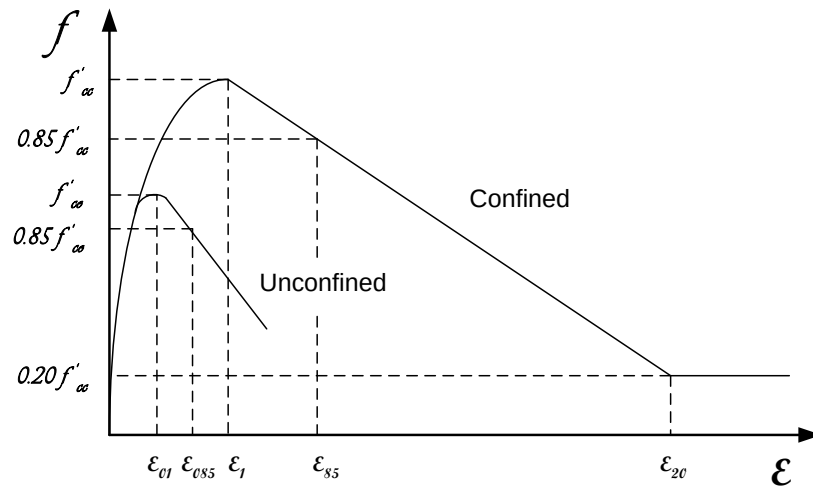
Column tests were used to verify the design procedure presented in Sec. 5.5.1 and 5.5.2 for column confinement and splice clamping. The verification include the comparison of inelastic deformation capacities of columns, as observed from experimentally recorded hysteretic relations with drift capacities computed using the design expression, Eq. 5.36. Solving for drift ratio  $\delta$ , from Eq. 5.36 results in the following:

$$\delta = \frac{t_j f_{Fj} \sqrt{k_c} P_o}{2D f'_c P} \quad [5.42]$$

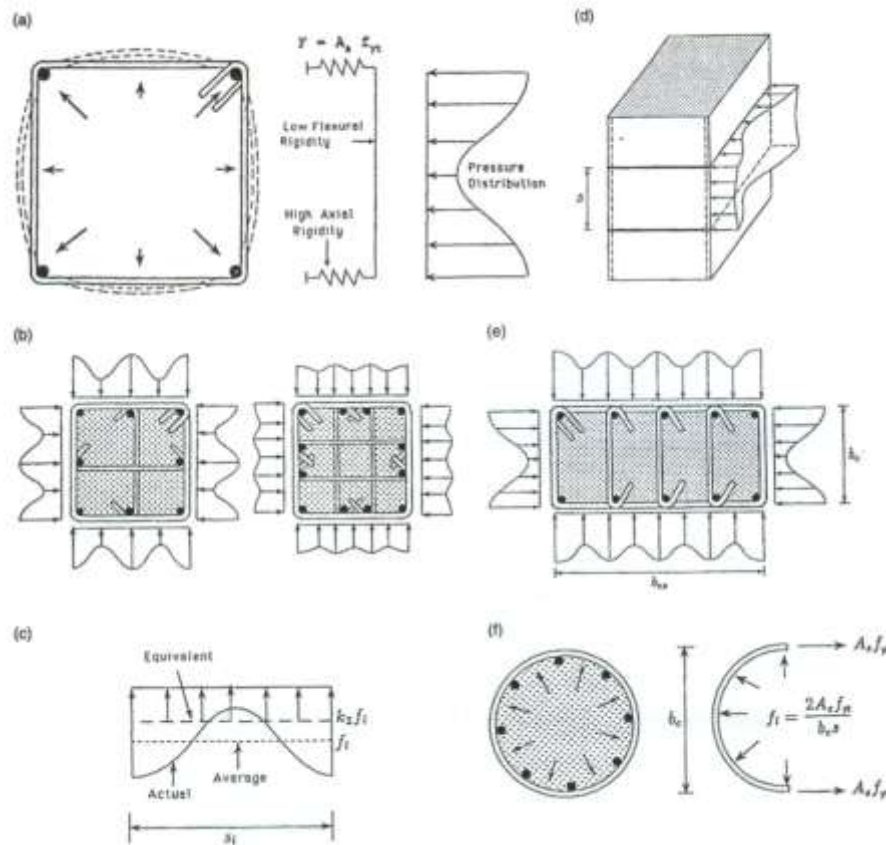
Substituting the values for all the design variables in Eq. 5.42, including the actual FRP thickness,  $t_j$  used in column specimens, one can compute the drift ratio predicted by the design equation. The computed drift capacities are shown in Fig. 5. 21 for all the columns tested in the current investigation, except for BR-SC-1, which is a square column with insufficient reinforcement splice length in the plastic hinge region for which the design expression does not apply. Fig. 5.21 indicates reasonably good predictions of drift capacities by the design expression. Recorded and computed drift capacities are shown in table 5.2. This validates the applicability of Eq. 5.36 to bridge columns. When used for actual design, Eq. 5.36 would be used to compute the required FRP thickness for the desired level of inelastic deformability as indicated by the design drift demand.

**Table 5.2** Recorded drift capacity vs computed drift capacity

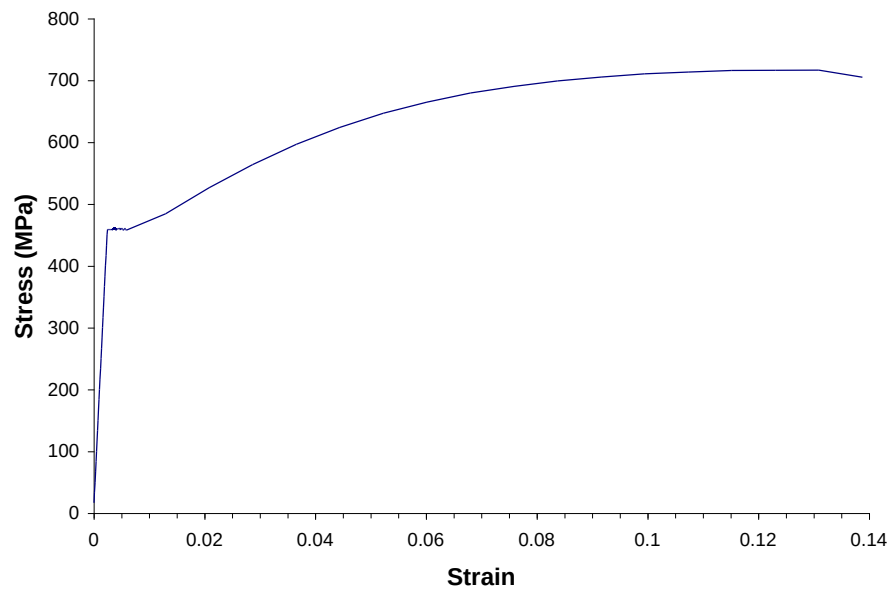
| <b>Column</b> | <b>Cross-section</b> | <b>Longitudinal Reinforcement</b> | <b>Retrofitting method</b> | <b>Recorded Drift %</b> | <b>Computed Drift %</b> |
|---------------|----------------------|-----------------------------------|----------------------------|-------------------------|-------------------------|
| BR-C8-2       | Circular             | Spliced, 20d                      | Wrapping,2 plies           | 4%                      | 3.0%                    |
| BR-C10-2      | Circular             | Spliced, 20d                      | Wrapping,2 plies           | 4%                      | 3.0%                    |
| BR-C6-1       | Circular             | Cont.                             | Wrapping,2 plies           | 8%                      | 6.5%                    |
| BR-SS-R       | Square               | Cont.                             | Wrapping,3 plies           | 6%                      | 4.5%                    |
| BR-CC-1       | Circular             | Spliced, 20d                      | Stay-in-Place FW           | 4%                      | 3.5%                    |
| BR-SC-2       | Square               | Spliced, 60d                      | Stay-in-Place FW           | 6%                      | 7.0%                    |



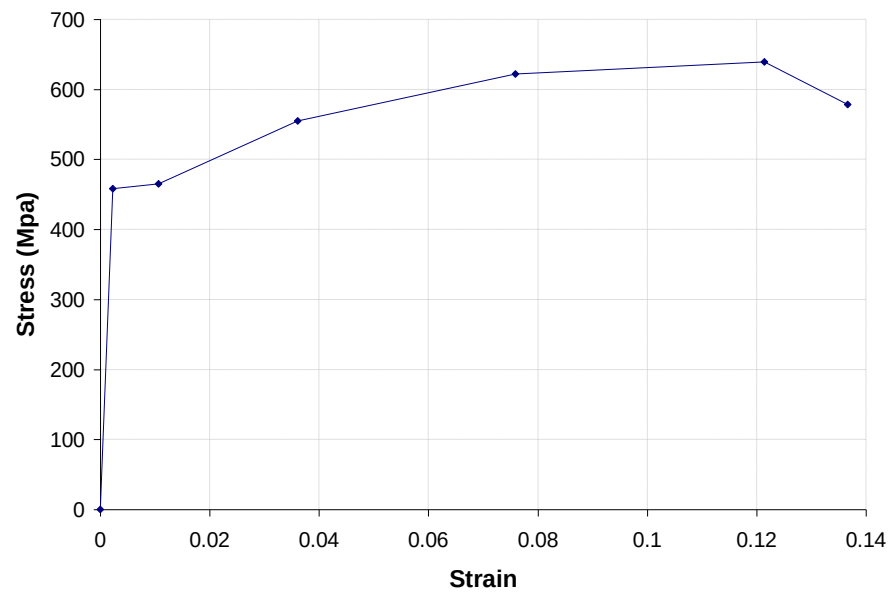
**Fig. 5.1** Stress-Strain Relationships for Unconfined and Confined Concrete  
(Razvi and Saatcioglu, 1999)



**Fig. 5.2** Confinement Pressures Induced by Different Arrangements of Column Reinforcement (Razvi and Saatcioglu, 1999)

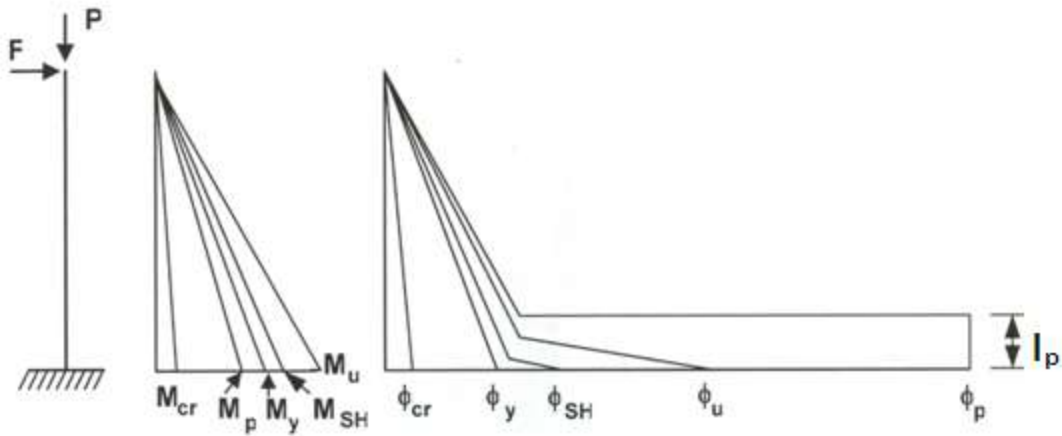


(a) Experimental stress strain relationship for #20M bars

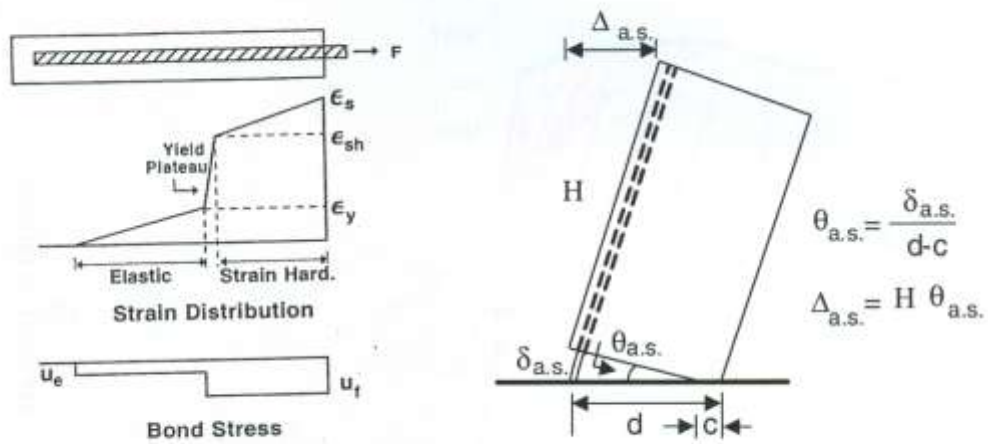


(b) Analytical model used for #20 M bars

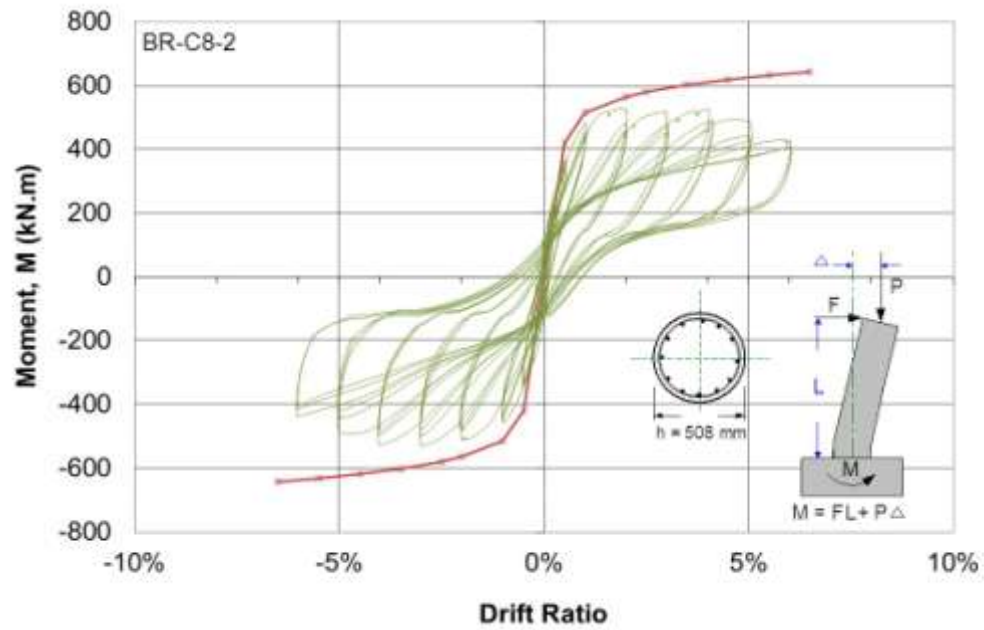
**Fig. 5.3** Stress-strain relationship for longitudinal reinforcement



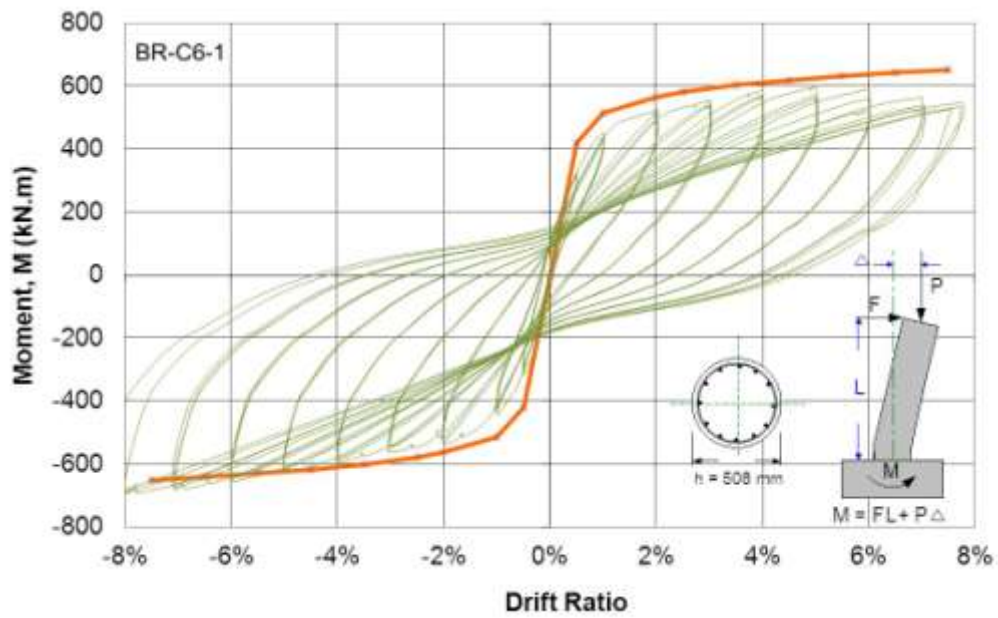
**Fig. 5.4** Progression of plastic hinge and distributions of curvatures at each of the control points for lateral drift calculations



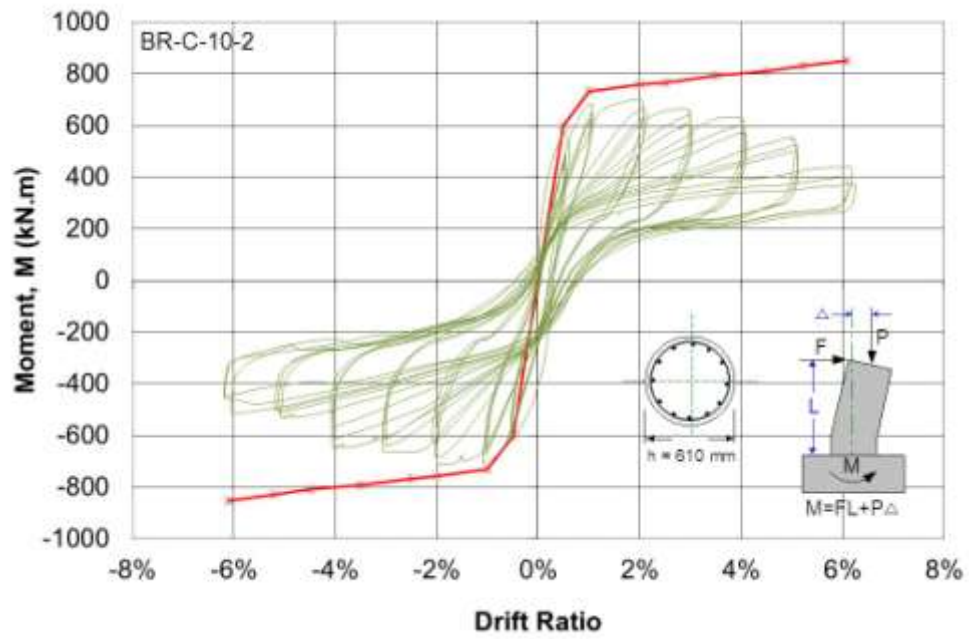
**Fig. 5.5** Typical Strain Distribution of an Embedded re-Bar with Elastic and Inelastic Segments and the Extension of Anchored Reinforcement



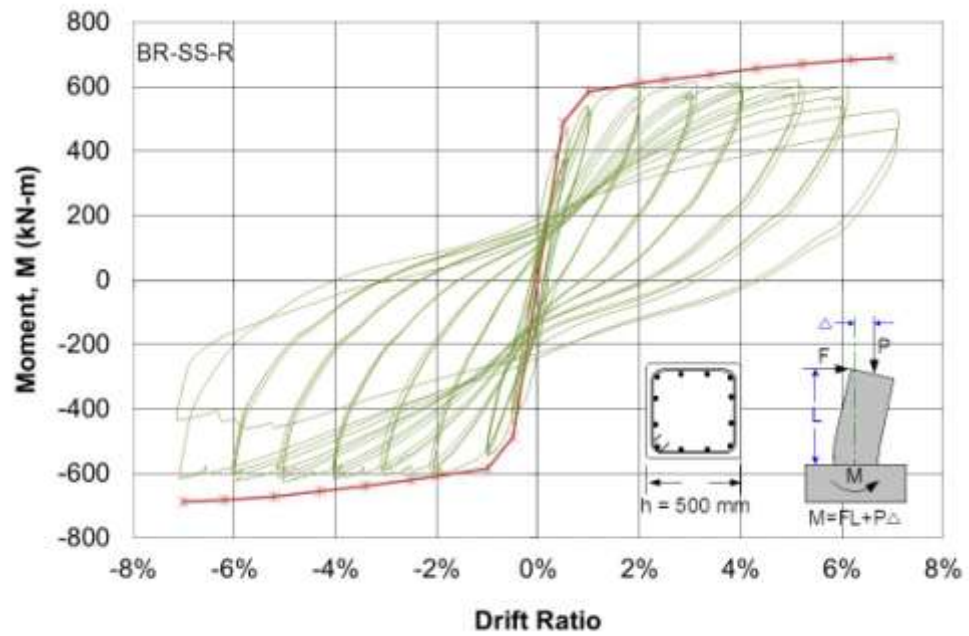
**Fig. 5.6** Analytical and experimental Moment-Drift relationships for column BR-C8-2



**Fig. 5.7** Analytical and experimental Moment-Drift relationships for column BR-C6-1



**Fig. 5.8** Analytical and experimental Moment-Drift relationships for column BR-C-10-2



**Fig. 5.9** Analytical and experimental Moment-Drift relationships for column BR-SS-R



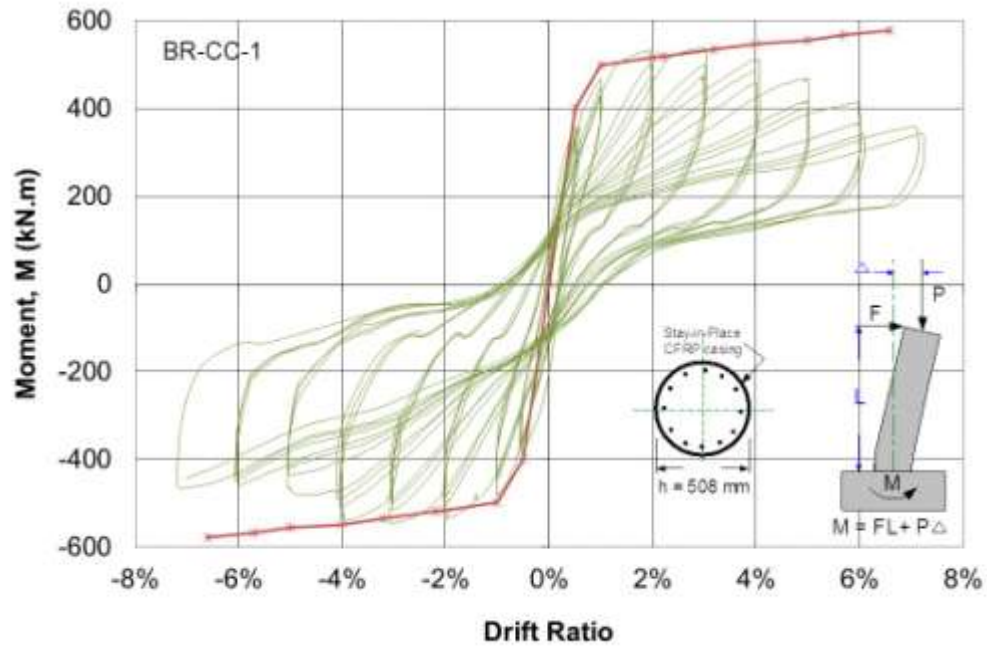


Fig. 5.10 Analytical and experimental Moment-Drift relationships for column BR-CC-1

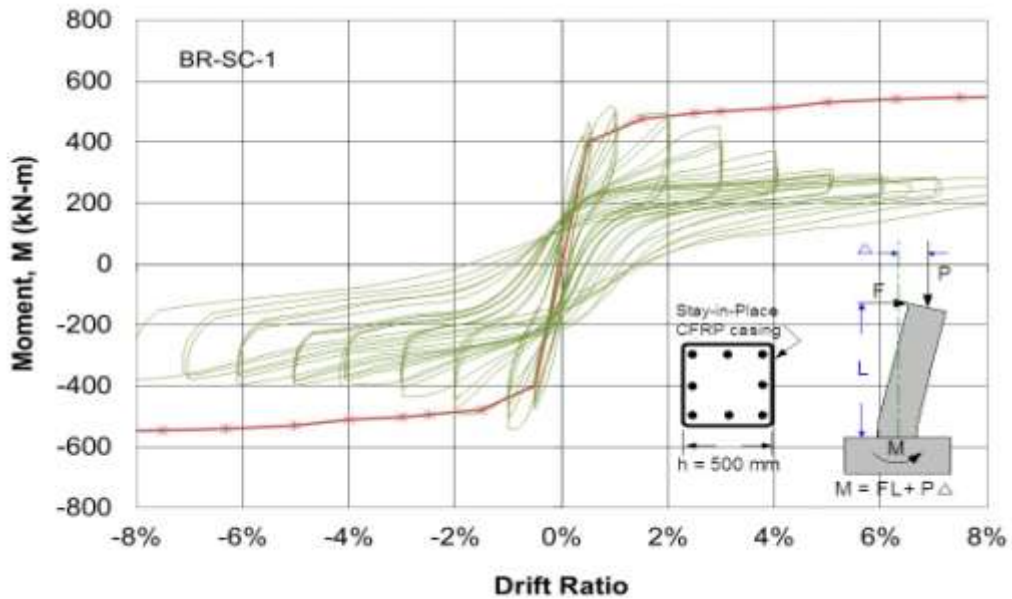


Fig. 5.11 Analytical and experimental Moment-Drift relationships for column BR-SC-1

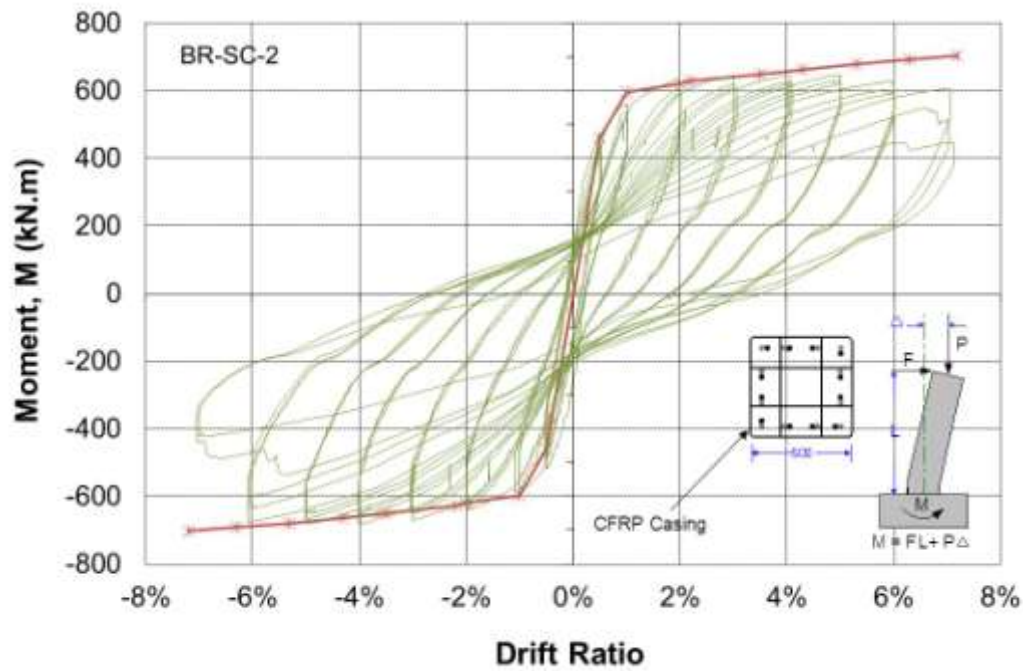


Fig. 5.12 Analytical and experimental Moment-Drift relationships for column BR-SC-2

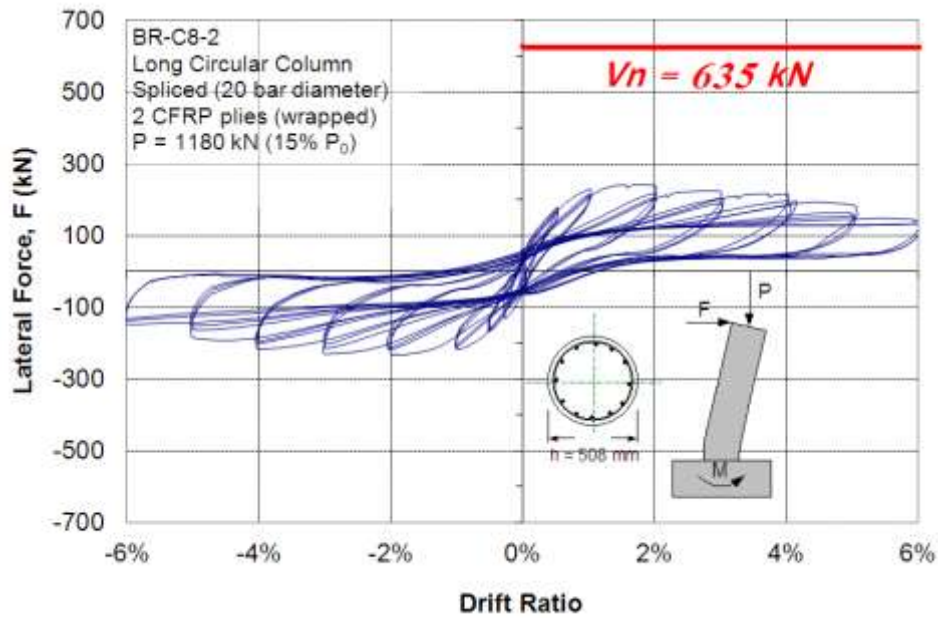


Fig. 5.13 Computed shear resistance against recorded data for column BR-C8-2

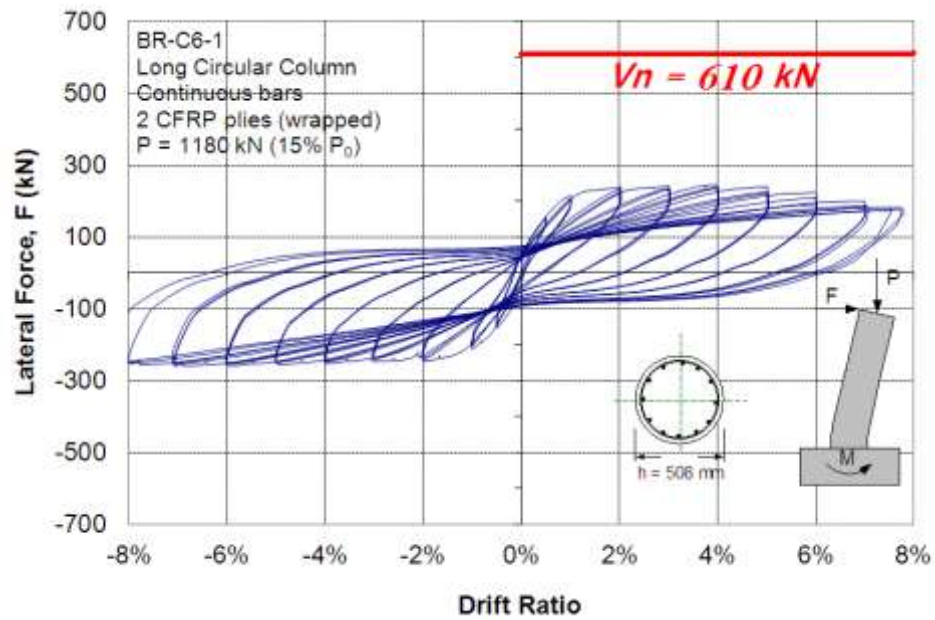


Fig. 5.14 Computed shear resistance against recorded data for column BR-C6-1

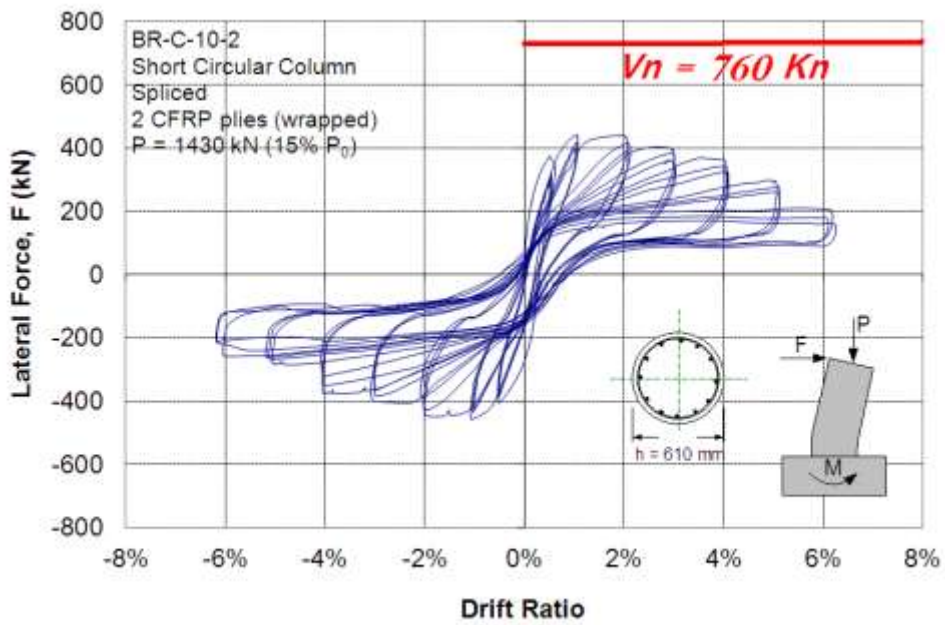


Fig. 5.15 Computed shear resistance against recorded data for column BR-C10-2

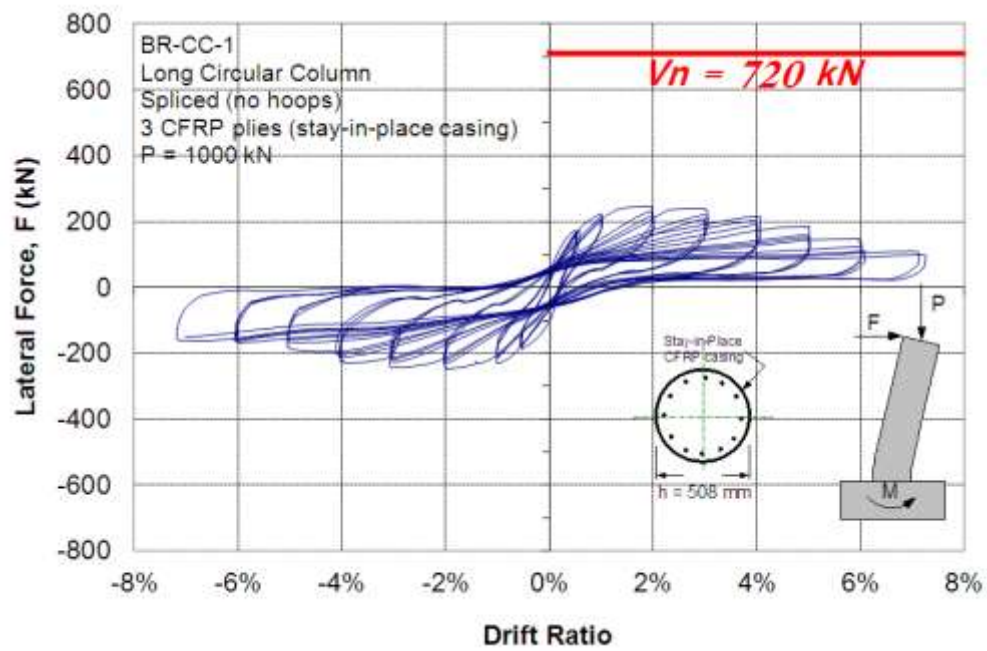


Fig. 5.16 Computed shear resistance against recorded data for column BR-CC-1

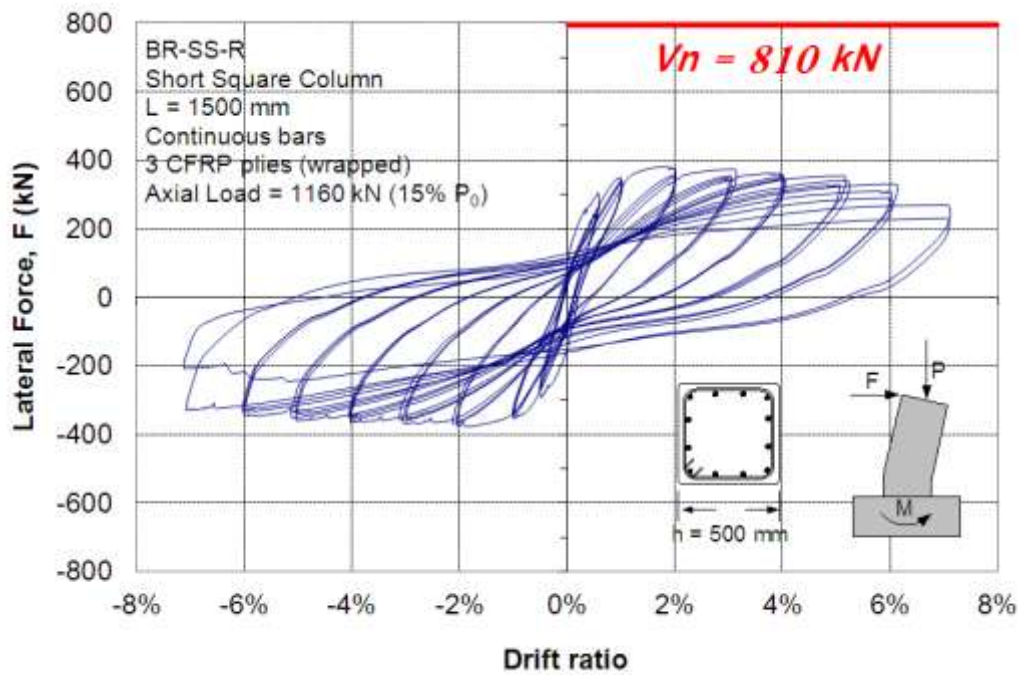


Fig. 5.17 Computed shear resistance against recorded data for column BR-SS-R



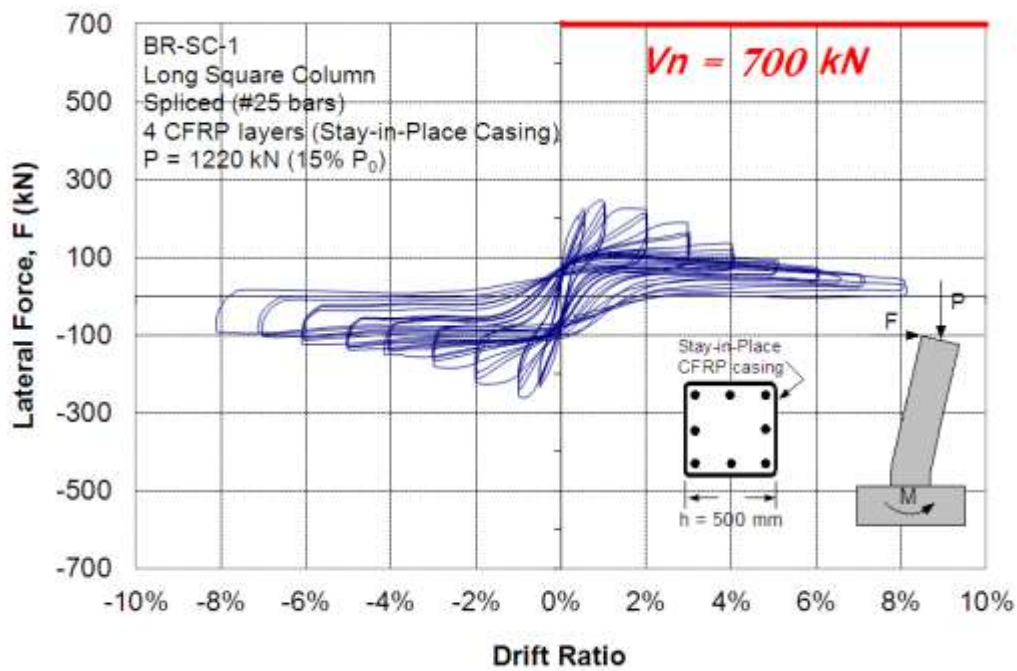


Fig. 5.18 Computed shear resistance against recorded data for column BR-SC-1

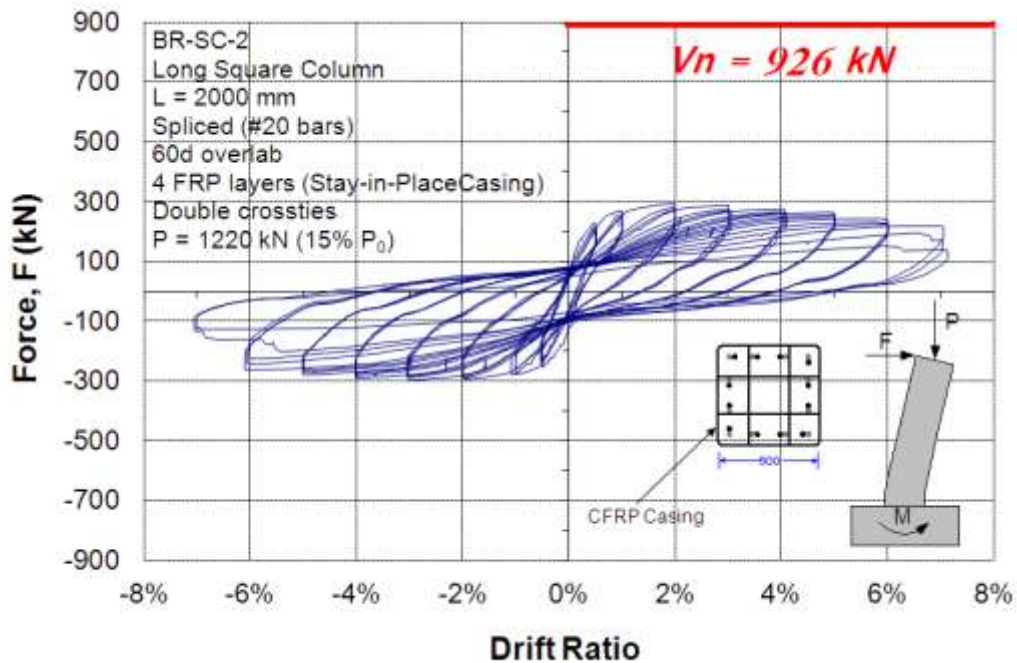
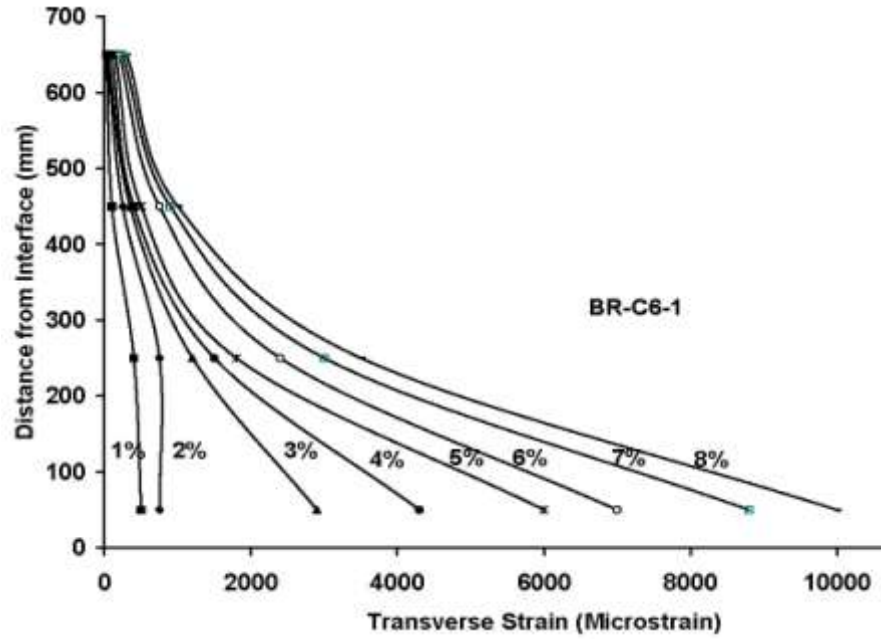
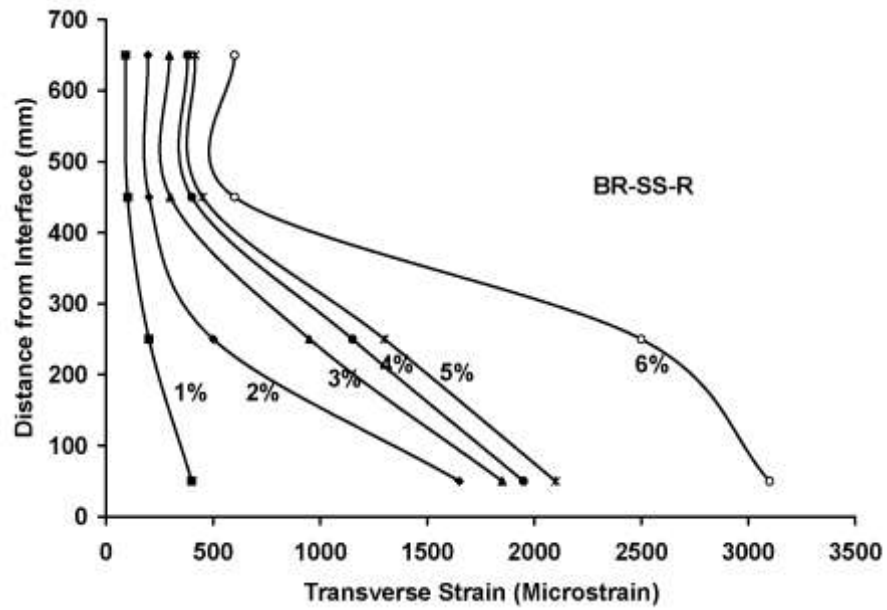


Fig. 5.19 Computed shear resistance against recorded data for column BR-SC-2

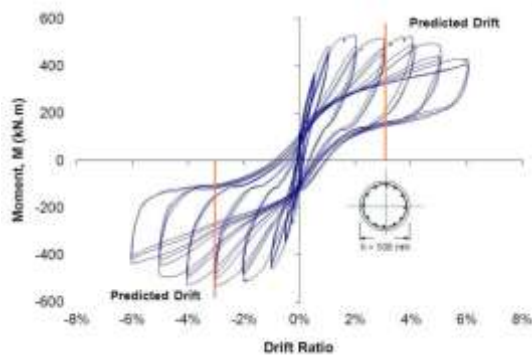


(a) Transverse FRP strains measured on circular column BR-C6-1 under 15%  $P_0$

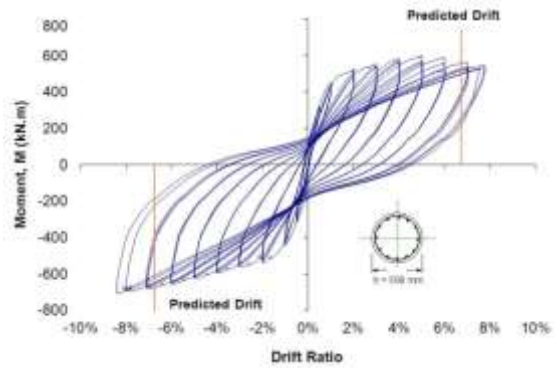


(b) Transverse FRP strains measured on square column BR-SS-R under 15%  $P_0$

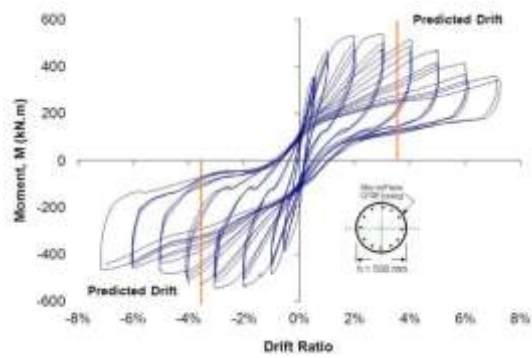
**Fig. 5.20** Transverse strain measurements on CFRP



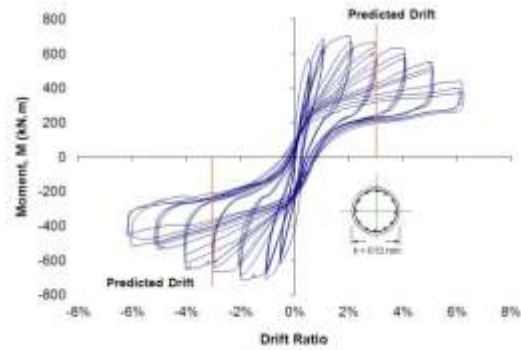
BR-C8-2



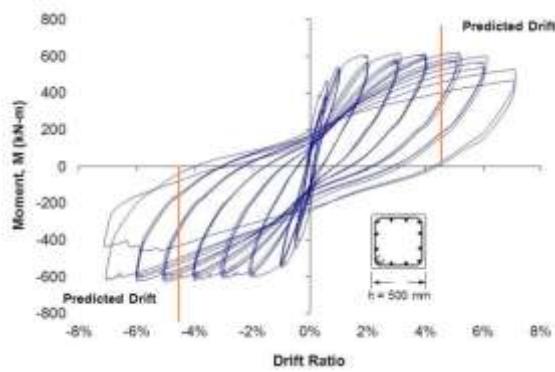
BR-C6-1



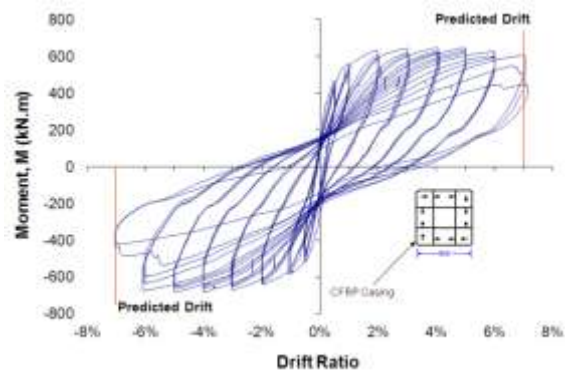
BR-CC-1



BR-C-10-2



BR-SS-R



BR-SC-2

Fig. 5.21 Predicted drift capacities

# CHAPTER 6

## SUMMARY AND CONCLUSIONS

### 6.1 GENERAL

A large proportion of current infrastructure in Canada was built prior to 1970's, using seismically deficient design codes. These structures pose significant seismic risk and need to be upgraded. Existing conventional column retrofit techniques pose economic, practical and structural challenges. Therefore, the use of FRP jacketing for existing seismically deficient concrete columns is currently being researched for infrastructure improvements. As well, the deteriorating bridge infrastructure provides the impetus for developing sustainable construction techniques. Use of FRP stay-in-place formwork is being considered for new bridge columns, with ability to speed up construction, reduce construction cost, protect concrete columns against corrosion and aggressive environments, while also improving structural performance as external transverse reinforcement. The current research has the objective of developing improved FRP jacketing for existing bridge columns, and FRP stay-in-place formwork for new columns. The following sections provide a summary of the research project, while also listing the conclusions drawn as a result of the research conducted.

### 6.2 SUMMARY

A combined experimental and analytical research project was carried out at the Structures Laboratory of the University of Ottawa. The experimental phase consisted of tests of full-size reinforced concrete bridge columns under simulated seismic loading. The primary objectives of the research program included; i) the development of an effective FRP jackets for retrofitting seismically deficient reinforced concrete columns, ii) the development of FRP stay-in-place formwork for new bridge columns. The analytical research consisted of column flexural and shear analysis to demonstrate the applicability of analytical techniques and material models in assessing strength and flexural deformation capacities of columns. A displacement-based design method was developed by modifying an existing method originally developed for columns reinforced with conventional steel reinforcement.



The column test specimens had either circular or square cross section, with cross-sectional dimensions ranging between 500 mm and 610 mm. Both flexure dominant columns with a shear span of 2.0 m, and shear –dominant columns with a shear span of 1.5 m were tested. The columns were subjected to a constant axial load equal to 15% of column concentric capacity, accompanied by incrementally increasing inelastic deformation reversals. Four of the seven columns tested were retrofitted using wet lay-up method with CFRP sheets. Different plies of CFRP were used to investigate the jacket thickness requirements. The effectiveness of CFRP jackets in improving column confinement and reinforcement splice clamping in circular and square columns were investigated. The columns were instrumented well with strain gauges and displacement transducers for FRP and steel reinforcement strain measurements, lateral displacement measurements, and rotation measurements in potential plastic hinge regions. Three of the columns tested were constructed using circular or square CFRP stay-in-place formwork, manufactured at the Structures Laboratory of the University of Ottawa. The square formwork had internally embedded FRP crossties to improve the effectiveness of FRP formwork. Column performance was measured and presented in the form of hysteretic relationships for moment-drift, force-drift and moment-rotation behavior. Displacements were measured at the column top, and rotations were measured at critical locations of columns near the foundation. Both flexural and anchorage slip rotations were measured and presented.

Analysis of columns included moment-curvature analysis of column sections with the effects of externally provided CFRP reinforcement. Concrete confinement model was introduced and member analysis was conducted using an assumed hinge length. Shear capacities of columns were computed based on 45-degree truss analogy and the applicable design expressions provided in CSA S806-2012. A displacement-based design procedure was developed for bridge columns with either retrofitted externally by FRP jackets or new column with stay-in-place FRP formwork. Drift ratio was used the displacement design parameter.

### **6.3 CONCLUSIONS**

The following conclusions can be drawn from the research project presented in this thesis:

1. CFRP jackets used to confine seismically deficient bridge columns are very effective in improving flexural deformability, resulting in 4% to 8% lateral drift ratios without significant strength loss, depending on the characteristics of columns.
2. Circular columns attain the desired level of ductility with lesser thickness of CFRP jackets, as compared to those for square columns.
3. While flexure-dominant columns with continuous longitudinal reinforcement in plastic hinge regions (or with sufficient lap length of 60 times the bar diameter) perform well, reaching the ductility and inelastic drift levels indicated above, columns with inadequately spliced longitudinal reinforcement may or may not benefit from CFRP jacketing depending on the cross-sectional shape and the amount of CFRP used in the jacket.
4. Circular columns having longitudinal reinforcement with a splice length of 20 times the bar diameter in the hinging region, can benefit from CFRP jackets and show ductile response if the amount of FRP is placed such that the transverse FRP strain is limited to 0.15%.
5. Square columns with splice deficiencies (splice length of  $20d_b$ ) cannot be retrofitted with CFRP jackets. The use CFRP jacket does provide some improvements, but cannot prevent the slippage of reinforcement and excessive pinching of hysteresis loops beyond 2 % drift. Therefore, it is not recommended to be used as a seismic retrofit strategy.
6. CFRP in lightly loaded bridge columns (axial compression of about 15% of concentric capacity) is effective as confinement reinforcement up to 0.5% strain in circular columns and 0.3% in square columns. These strain values are recommended as average design values in computing the effective material strength. The effectiveness of FRP composites decreases in square columns due to the non-uniform confinement pressure and inability to develop full hoop tension.
7. Surface bonded CFRP sheets provide effective crack control against shear-induced diagonal tensile stresses, and improve column shear resistance. CFRP jackets were able to change the mode of behavior from shear–dominant brittle response to flexure-dominant ductile response. The column wrapping provided, with a 100 mm overlap, ensured perfect bond. There was no debonding or delamination observed during testing.

8. CFRP stay-in-place formwork provides column response that is similar to that observed in CFRP wrapped columns. CFRP tube used in the circular flexure-dominant column that was tested in the experimental program developed a lateral drift capacity of 4% even with insufficient reinforcement splice length of  $20d_b$  and without internal steel ties.
9. Square CFRP stay-in-place formwork, when used with internal crossties and sufficient longitudinal reinforcement splice length show ductile response. The square column with double internal crossties and  $60d_b$  splice length developed 6% lateral drift without strength decay.
10. The concrete cover of 25 mm provided inside the CFRP stay-in-place forms, combined with the rigidity of the FRP shell, provided sufficient restraint against the buckling of compression reinforcement, even without the use of internal steel ties.
11. The concrete confinement model developed by Saatcioglu and Razvi (1992), modified to account for the presence of continuous CFRP composite, can be used to compute column strength and inelastic deformability.
12. Plane section analysis employed for constructing moment-curvature relationship provides accurate estimation of column strength when used with an appropriate concrete confinement model.
13. Plastic member analysis, utilizing the moment-curvature analysis and an assumed constant hinge length provides excellent predictions of inelastic deformations if the behavior is not dominated by reinforcement slip or shear.
14. Shear strength expressions given in CSA S806-2012 can be used to assess the nominal shear capacity of FRP retrofitted columns.
15. The displacement-based design approach developed for FRP column retrofitting and FRP stay-in-place formwork can be used effectively in designing bridge columns. The design expression can be used to predict column drift capacity for given FRP configuration and material properties.

## **6.4 RECOMMENDATIONS FOR FUTURE RESEARCH**

The current research projects provides experimental and analytical research data that provides better understanding of bridge column response under seismic loading when retrofitted with

CFRP jackets and when cast in CFRP stay-in-place formwork. Additional research is needed to expand the knowledge in this area. Specifically, the following tasks are suggested for future research:

1. Additional column tests under different levels of axial compression and cross-sectional configurations, including tests of rectangular columns with lateral load reversals applied in short and long cross-sectional directions.
2. Column tests with different splice lengths, as the current research covers only the old practice of using 20 times the bar diameter.
3. Use of glass FRP jackets and stay-in-place formworks for similar size bridge columns.
4. Further testing of square and rectangular columns with different configuration of internal FRP crossties, having different fiber contents.
5. Testing of square columns considering the effect of rounded corner as a design parameter.
6. Analytical modeling of spliced reinforcement in FRP confined concrete bridge columns.
7. Analysis of columns, incorporating the buckling of compression reinforcement.

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# Wabo®MBrace

## Composite Strengthening System



### DESCRIPTION:

The Wabo®MBrace Composite Strengthening System is an externally bonded FRP (fiber reinforced polymer) reinforcement system used to strengthen existing concrete and masonry structures. Watson Bowman Acme offers a fabric version as well as a carbon/epoxy laminate version of Wabo®MBrace Composite Strengthening System.

The field-proven Wabo®MBrace fabric technology consists of high performance dry fabrics constructed of aerospace-grade carbon, aramid, and E-glass fibers plus polymer resins formulated specifically for the Wabo®MBrace system. The system specific polymer resins include a surface primer and putty filler to insure proper bond of the system and saturating resins to wet out the fiber fabrics and form a high performance FRP laminate. The polymer resins can also be provided in a vapor permeable version (referred to as Resicem), which allows moisture to permeate through the Wabo®MBrace system.

The Wabo®MBrace S&P Laminate is a high strength pre-manufactured carbon/epoxy laminate. These laminates are used for surface mounted or near surface mounted applications providing additional strength and stiffness to concrete and masonry structures.

### RECOMMENDED FOR:

The Wabo®MBrace System can be used in a wide variety of strengthening applications.

- Structural Upgrade or Change in Use of a Concrete or Masonry Structure
- Restoration of Structural Capacity lost to Damage or Deterioration of a Concrete Structure
- Correcting Errors in the Original Construction of a Concrete or Masonry Structure
- Seismic Retrofit of Concrete and Masonry Columns and Walls
- Blast Upgrades/Mitigation
- Improving Impact or Wear Resistance of Structural Members
- General Infrastructure Protection and Repair
- Crack Control
- Strengthening of steel and timber members is also possible



### FEATURES/ BENEFITS:

- High Strength
- Lightweight
- Highly durable, non-corrosive
- Low installation time
- Can be installed in areas of limited access
- Easy to conceal, minimal change to existing member dimensions
- Forms around complex surface shapes

### APPLICATION

The Wabo®MBrace fabric based system is installed by a technique known as wet lay-up. This technique involves applying the lightweight, flexible fiber fabrics onto a prepared surface of a structural member using uncured polymer resins. Once the resins cure, the result is a high strength bonded FRP laminate that is similar to bonding steel plates to the surface. Unlike bonded steel plates, however, the Wabo®MBrace system can be easily installed in a short time period with minimal labor and in areas of difficult access. Additionally, the Wabo®MBrace system offers not only improved structural performance, but also a durable reinforcement system that can act to protect the underlying structural member.

The Wabo®MBrace S&P Laminates are bonded with a liquid or paste adhesive directly to the prepared surface of concrete or masonry.

#### **WABO@MBRACE EPOXY COMPONENTS:**

- **Wabo@MBrace Primer** – A low viscosity epoxy material that penetrates the pore structure of the substrate
- **Wabo@MBrace Putty** – A thick epoxy paste used to fill in small surface defects in the substrate
- **Wabo@MBrace Saturant** – A sag resistant epoxy impregnation resin used to wet out the fiber fabrics
- **Wabo@MBrace Saturant LTC** – An alternative impregnation resin that can cure in lower temperature environments

#### **WABO@MBRACE RESICEM EPOXY COMPONENTS:**

- **Wabo@MBrace Resicem Primer** – A low viscosity moisture permeable epoxy material that penetrates the pore structure of the substrate
- **Wabo@MBrace Resicem Putty** – A thick moisture permeable epoxy paste used to fill in small surface defects in the substrate
- **Wabo@MBrace Resicem Saturant** – A moisture permeable sag resistant epoxy impregnation resin used to wet out the fiber fabrics
- **Wabo@MBrace Resicem Saturant LTC** – A moisture permeable alternative impregnation resin that can cure in lower temperature environments

#### **WABO@MBRACE FIBER REINFORCEMENTS:**

- **Wabo@MBrace CF 130** – A 9oz unidirectional, high strength carbon fiber fabric
- **Wabo@MBrace CF 160** – An 18oz unidirectional, high strength carbon fiber fabric
- **Wabo@MBrace CF 530** – A unidirectional, high modulus carbon fiber fabric
- **Wabo@MBrace AK 60** – A unidirectional, high performance aramid fiber fabric constructed of DuPont Kevlar® brand fibers
- **Wabo@MBrace EG 900** – A unidirectional, E-glass fiber fabric

#### **WABO@MBRACE S&P LAMINATES:**

- **Wabo@MBrace S&P LAMINATE 10/1.4** – A pre-fabricated carbon/epoxy laminate utilized for near surface mounted applications.
- **Wabo@MBrace S&P LAMINATE 50/1.4** – A 2-inch wide pre-fabricated carbon/epoxy laminate utilized for surface mounted applications.

- **Wabo@MBrace S&P LAMINATE 100/1.4** – A 4-inch wide pre-fabricated carbon/epoxy laminate utilized for surface mounted applications.

#### **WABO@MBRACE TOPCOATS:**

- **MBrace Topcoat ATX** – An acrylic topcoat that matches the aesthetic appearance of bare concrete and provides protection against UV light.
- **MBrace Topcoat FRL** – An intumescent coating that dramatically reduces the flammability and smoke generation of the Wabo@MBrace system. For use in occupied enclosed building environments.
- **MBrace Topcoat URE** – A low VOC, two component, aliphatic, protective urethane topcoat for use where moderate to severe environmental conditions exist. Suitable for heavy industrial and marine atmospheres and where long maintenance free service life is required.

#### **STORAGE AND HANDLING:**

Store in a cool, dry area [50 to 90 °F (10 to 32 °C)] away from direct sunlight, flame, or other hazards.

Wabo@MBrace Fiber Reinforcements contain carbon, glass, and/or aramid fibers. While handling Wabo@MBrace Fiber Reinforcements, wear appropriate work clothing to minimize contact.

Product Material Safety Data Sheets (MSDS) are available and should be consulted and on hand whenever handling these products.

These products are for professional and industrial use only and are only installed by trained and qualified applicators. Trained applicators must follow installation instructions.

#### **MAINTENANCE:**

Periodically inspect the applied material and repair localized areas as needed. Consult a Wabo@MBrace representative for additional information.

Visit us on the web for the most current product information and news:

**[www.wbacorp.com](http://www.wbacorp.com)**

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#### **LIMITED WARRANTY:**

Watson Bowman Acme Corp. warrants that this product conforms to its current applicable specifications. WATSON BOWMAN ACME CORP. MAKES NO OTHER WARRANTY, EXPRESS OR IMPLIED, INCLUDING ANY WARRANTY OF MERCHANTABILITY OR WARRANTY OF FITNESS FOR A PARTICULAR PURPOSE. The sole and exclusive remedy of Purchaser for any claim concerning this product, including, but not limited to, claims alleging breach of warranty, negligence, strict liability or otherwise, is the replacement of product or refund of the purchase price, at the sole option of Watson Bowman Acme Corp. Any claims concerning this product shall be submitted in writing within one year of the delivery date of this product to Purchaser and any claims not presented within that period are waived by Purchaser. IN NO EVENT SHALL WATSON BOWMAN ACME CORP. BE LIABLE FOR ANY SPECIAL, INCIDENTAL, CONSEQUENTIAL (INCLUDES LOSS OF PROFITS) OR PUNITIVE DAMAGES.

Other warranties may be available when the product is installed by a Wabo-certified installer. Contact your local Wabo representative for details.

The data expressed herein is true and accurate to the best of our knowledge at the time published; it is, however, subject to change without notice.

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