# DYNAMIC RESPONSE ANALYSIS OF PILOTIS BUILDING WITH RETROFITTED PILOTIS FRAMES

耐震補強されたピロティ建築物の地震応答解析

**Technical report of Architectural Institute of Japan (AIJ)**
**Kyushu-Okinawa branch, Japan**
**(Research grant for young researchers in 2006)**

**MD. NAFIUR RAHMAN**
**PASHA JAVADI**

**University of the Ryukyus**
**Department of Civil Engineering and Architecture**

**May 2007**

成果報告書和文概要

## 耐震補強されたピロティ建築物の地震応答解析

ピロティ建築物は上階に比べて水平耐力や水平剛性が小さい第1層を有する建築物のことである。過去の地震において，ピロティ建築物では第1層（ピロティ層）に損傷が集中したために，過大な変形あるいは大破が生じた例が多数観測されている。ピロティ層の水平耐力，剛性および靭性を増大させるために，ピロティ建築物への合成極厚無筋壁補強法が提案されている。この合成極厚無筋壁補強法は，緊張PC鋼棒を使用して増し打ちコンクリート表面に鋼板を圧着した袖壁付き柱やオープンフレームにより研究が行われた。本論文では，提案された補強法を適用する前と適用した後の既存RCピロティ建築物の非線形地震応答解析を行い，層せん断力や層間変形角の比較を行う。

ピロティ建築物の非線形地震応答解析では，2次元のRUAUMOKOプログラムを使用する。解析を行うに際し，合成極厚無筋壁補強が適用された柱は，鉛直および水平方向の2つの独立な構成要素を有するバネ要素としてモデル化され，履歴則には実験結果が反映される。曲げ挙動をモデル化するのに一般的によく使用される有名な履歴則の一つに修正武田モデルがある。しかしながら，この履歴則はピンチング効果を考慮していない。ピンチング効果を考慮するためにSINA履歴則を使用する。このSINA履歴則のパラメータのいくつかは，片袖壁付き柱部材の実験結果に基づいて修正される。実験結果と解析結果の適合性を向上させるために修正されたSINA履歴則のパラメータは，固定値である1/2から変数αへと修正された除荷べき指数とピンチングポイントの変位座標を制御するために定義されたピンチング係数である。

非線形地震応答解析は既存のRCピロティ建築物（4層，梁間方向1スパン，桁行方向4スパン）について行われる。理想化されたモデルでは，梁と床は曲げに対して剛体であるとしており，梁と柱の軸方向変形は無視されている。主な構成要素である柱と耐震壁は柱梁要素とバネ要素として計算される。建物の全体応答を検証するために，各層が両端ピンの剛棒で接続された連続する5つの骨組として建物はモデル化されている。骨組の基礎は固定されており，基礎の回転は無視されている。柱と耐震壁の非線形挙動をモデル化するために，修正武田モデルと原点指向型履歴モデルが使用されている。解析には3つの地震波（Taft，Hachinohe，El Centro）を使用した。耐震補強前では，柱の降伏モーメントは日本建築防災協会によるせん断強度計算値に近く，それゆえ，修正武田モデルに代わり原点指向型履歴モデルを使用し，耐力低下則の定義を行った。解析の結果，開始後何秒か後の3地震波ではフレーム5の耐震壁にせん断破壊が最初に発生し，それから当該フレームの柱にもせん断破壊が発生した。その結果，ピロティ層が突然崩壊に至った。ピロティ層のせん断力が水平耐力に達したとき，解析は終了を迎えた。

ピロティ層の崩壊を防止するためにはピロティ層の柱の水平耐力，剛性および靭性を増大させる耐震補強が必要である。ピロティ層の耐震補強においては，合成極厚無筋壁補強法およびPC鋼棒による靭性型補強法の組み合わせが有効である。本補強計画では，合成極厚無筋壁補強法により水平耐力，剛性および靭性を増大させ，一方，PC鋼棒による外部横補強法により破壊モードをせん断から曲げへ変化させることで，靭性能を改善させることができる。耐震補強法の選択では，平面計画における水平剛性の非対称は建物のねじりを防止するために避けなければならない。解析の結果，提案した合成極厚無筋壁補強法の適用後，ピロティ層の水平耐力が増大したことにより建物は崩壊を免れた。ピロティ層の層間変形角は，許容値内（1%）である。さらに，第2層のせん断力応答値は第2層の崩壊を防止するのに十分小さい。

## ABSTRACT

A pilotis-type building is a building that its first story has less lateral strength and stiffness compared to upper stories. In past earthquakes, it was observed that in this type of building the large deformation or complete collapse occurred due to the damage concentrated in the first story. To increase the lateral strength, stiffness and ductility of first story columns, a retrofit technique of combination of PC bar and thick hybrid wall that previously investigated for shear critical column and wing-wall column, respectively, is proposed for pilotis building. In this paper, nonlinear dynamic analysis of an existing pilotis-type RC building before and after employing the proposed retrofit system are conducted, and their story shear force and drift angle responses are verified.

For the nonlinear dynamic analysis of the pilotis building, the computer program RUAUMOKO-2D is used. To conduct the analysis, the columns retrofitted by thick hybrid wall and PC bar are modeled as a spring element with two independent components in the vertical and the horizontal directions, in which a suitable hysteresis rule should be found according to experimental result. One of the famous hysteresis rules that is commonly used for modeling the flexural behavior is the modified Takeda rule, however, this rule can not consider the pinching effect. To take into account this effect, the SINA hysteresis rule is used with modifying some of its parameters based on the hysteresis behavior of retrofitted shear critical column and one-sided wing-wall column. To achieve the best agreement between the experimental and the analytical result, the parameters of SINA rule that are modified are the unloading power factor that is changed from a fixed value of ½ to a changeable value of á, and the pinching factor that is defined to control the displacement coordinate of the pinching point.

Nonlinear dynamic analyses are conducted for an existing four-story pilotis-type RC building with one bay in one direction and four bays in another direction. In the idealized model, the beams and floor systems are considered to be rigid in flexure, and the axial deformations of beams and columns are neglected. The main components, namely columns and shear walls are simulated as beam-column and spring elements, respectively. To verify the global response of the building, it is modeled as five frames that are laid in series and connected with rigid links at each story level. The bases of frames are assumed to be fixed and the uplift of foundation is neglected. The modified Takeda and origin centered hysteresis rules are used to model the nonlinear behavior of nonretrofitted column and shear wall, respectively. Three earthquake records (namely, Taft, Hachinohe, and El Centro) are used to analyze the building. Before retrofitting, since yield moments of columns are close to their calculated shear strength according to JBDPA guidelines, the origin centered rule is considered in place of the modified Takeda rule, and the strength degradation pattern is defined. The analytical results show that for all three earthquakes after few seconds, the shear failure in the first story shear wall of frame-5 is occurred first and then that occurred in columns of that frame. As a result, the sudden collapse is occurred in the first story.

To prevent the collapse in the first story of the pilotis building, seismic retrofitting of shear-critical columns in that story is necessary so that their lateral strength, stiffness and ductility are increased. In retrofitting the first story columns, a combination of using the techniques of thick hybrid wall system and only the PC bar prestressing can be effective. In this retrofit plan, the thick hybrid wall system increases the lateral strength, stiffness and ductility, while only PC bars prestressing improves the ductility by changing the failure mode from shear to flexural one. During the selection of retrofit, the asymmetric stiffness in plan is avoided to eliminate the torsional effect. The analytical results show that after employing the proposed retrofit method, the first story failure is prevented by increasing the lateral strength of that story. The story drift response of that story is also within the allowable limit (namely, 1%). Moreover, the shear force response of the second story is adequate to prevent the failure in that story.

# ACKNOWLEDGMENTS

The authors would like to express profound gratitude and sincere thanks to Tetsuo Yamakawa, Professor of Department of Civil Engineering and Architecture, Faculty of Engineering, University of the Ryukyus, for his appreciable inspiration during this research work. The authors wish to acknowledge A. J. Carr, Professor of Department of Civil Engineering, College of Engineering, University of Canterbury, New Zealand for his valuable guidance and advice regarding the modelling and dynamic analysis using "RUAUMOKO" software.

The investigation reported herein was carried by research grant for young researchers from the Architectural Institute of Japan (AIJ), Kyushu-Okinawa branch, Japan, 2006. The authors would like to give thanks to Mr. Kozo Nakada, Research associate, Shinya Takara, doctoral student and, Koki Maeda master's student, Department of Civil Engineering and Architecture, Faculty of Engineering, University of the Ryukyus, for their valuable supports.

# TABLE OF CONTENTS


Page

ABSTRACT ....................................................................................................................i

ACKNOWLEDGMENTS ...................................................................................... iv

TABLE OF CONTENTS .......................................................................................v

LIST OF FIGURES ..............................................................................................vi

LIST OF TABLES ...............................................................................................vii

Chapter 1 INTRODUCTION ..............................................................................1

Chapter 2 BACKGROUND OF PROPOSED RETROFIT TECHNIQUE................................2

Chapter 3 CALIBRATION OF HYSTERESIS RULES ...........................................9

3.1 Common flexural hysteresis rules ...........................................................9

3.2 Calibration of hysteresis rule of the column retrofitted by PC bar prestressing..........10

3.3 Calibration of hysteresis rule of the column retrofitted by thick hybrid wall...............12

Chapter 4 DYNAMIC ANALYSIS OF AN EXISTING PILOTIS BUILDING........................14

4.1 Details of an existing pilotis building.....................................................14

4.2 Modeling of the pilotis building............................................................15

4.3 Modal analysis of the pilotis building.....................................................16

4.4 Input earthquake waves.......................................................................17

4.5 Dynamic responses of the pilotis building before retrofitting..................................17

4.6 Dynamic analysis of the pilotis building retrofitted by ductility-type technique.........18

4.7 Dynamic analysis of pilotis building retrofitted by strength-ductility-type technique........................................................................................21

4.8 Discussion about retrofit procedure ......................................................21

4.9 Conclusions ........................................................................................23

REFERENCES .......................................................................................................24

PUBLICATION.......................................................................................................24

# LIST OF FIGURES

Page

1 Mechanism of pilotis building ....................2

2.1 Example of conventional and proposed retrofit techniques....................5

2.2 An example of plan and elevation of retrofitted pilotis building and various retrofit styles at different locations of column ....................6

2.3.1 Test result of specimen R03WO-P0 ....................7

2.3.2 Test result of specimen R03WO-S ....................7

2.3.3 Test result of specimen R02WC-P0 ....................7

2.3.4 Test result of specimen R03WC-P200S....................8

2.3.5 Test result of specimen R05P-P0....................8

2.3.6 Test result of specimen R05P-OR. ....................8

2.3.6 Test result of specimen R05P-WDB ....................9

3.1 Hysteresis rules for modeling the flexural behavior....................9

3.2 Test setup and loading program....................11

3.3 Modified SINA model....................12

3.4 Simulated specimen....................12

3.5 Comparison of test and analytical results of column retrofitted by PC bar prestressing....................12

3.6 Comparison of test and analytical results of wing-wall column retrofitted by thick hybrid wall...13

4.1 First story paln and elavations of an existing pilotis building....................14

4.2 Modeling of the pilotis building....................15

4.3 Mode shapes of the pilotis building....................16

4.4 Response of the first story shear wall at frame 5 before retrofit....................18

4.5 Response of the first story column at frame 2 before retrofit....................18

4.6 Drift angle response of the first story before retrofit....................18

4.7 Retrofiting column by PC bar prestressing....................19

4.8(a) Maximum drift angle of the stories(before retrofit)....................20

4.8(b) Comparision of the first story response between modified SINA and Takeda rule before retrofit....................20

4.9 Hysteresis respose of the first story in ductility-type technique....................20

4.10 Comparison of first story response in ductility-type technique....................20

4.11 Details of strength-ductility-type retrofit....................22

4.12 Drift angle and shear force of the stories in strength-ductility-type retrofit....................22

4.13 Time history response of first story during the retrofit procedure....................23

4.14 Hysteresis response of first story during the retrofit procedure....................23

# LIST OF TABLES

Page

3.1 Comparision between original column and retrofitted one by PC bar ......................................11

3.2 Non-retrofitted and retrofitted wing-wall column.....................................................................13

4.1 Details of columns....................................................................................................................14

4.2 Details of beams.......................................................................................................................15

4.3 Results of modal analysis.........................................................................................................17

4.4 Intensity of original and scaled earthquake records..................................................................17

# Chapter 1

# INTRODUCTION

The pilotis-type buildings (i.e., buildings with a first soft-story) especially low-rise to mid-rise RC frame buildings, such as school buildings, dwelling houses, police stations, and hospitals are very customary in the urban areas to provide adequate open spaces for parking or good amenity through ventilation in the first story. In these buildings, the first story is intentionally made taller to create sufficient open spaces and the structural walls (namely, shear walls) that provided in the upper stories are discontinued to meet the change in the first story. Moreover, the non-structural walls, such as spandrel-walls, wing-walls besides the columns, are usually neglected in the practical structural analyses and designs though they can contribute significantly to the strength and stiffness of the framing system. The investigations and observations after past earthquakes, in particular from the 1995 Hyogoken-Nanbu Earthquake in Japan have revealed that many of the pilotis buildings designed with both older and updated design codes had suffered the extensive structural and non-structural damages. Most of the damages were concentrated on the first story due to the abrupt change in lateral strength and stiffness. Although, the presence of various kinds of walls inadvertently increases the lateral strength, stiffness and energy dissipation capacity of stories above the first story, this generally creates a structural vertical discontinuity of the stiffness and strength which can cause the formation of a so-called soft-story mechanism in the first story during earthquake. By the abrupt reduction in lateral strength and stiffness in the first story, the earthquake induced deformations tend to concentrate in that weak story and the high story drift demand of an earthquake motion often leads to the failure of soft story columns and eventually the collapse of the building. Again, for large story displacement, there is a risk of collapse of the pilotis building by the $P$-$\delta$ effect, which reduces the story shear capacity of the 1st pilotis story due to the additional overturning moment developed by the large story displacement. Moreover, during an earthquake, the exterior columns, especially corner columns are usually subjected to high axial compression force due to the overturning moment by the horizontal shear force, resulting in flexural compression failure at a relatively small deformation.

As a reference for the case of pilotis building, the qualitative load-deformation relationship and the mechanism of such building based on the T. Paulay's "Capacity Design" philosophy (Paulay T. 1979) comprised of ductile link as the first pilotis-story with the bare frame and brittle links as the stories above the first story with framed shear wall are illustrated in **Fig. 1**. Since the lateral strength of pilotis-story is low, the energy absorption of pilotis building is controlled by the first pilotis-story. According to Newmark's equal-energy principle (Newmark NM. and Rosenblueth E. 1971), the building with low strength requires the high ductility demand. The overall ductility demand is much more readily achieved when plastic hinges develop in all the beams instead of only in the first-story column. The

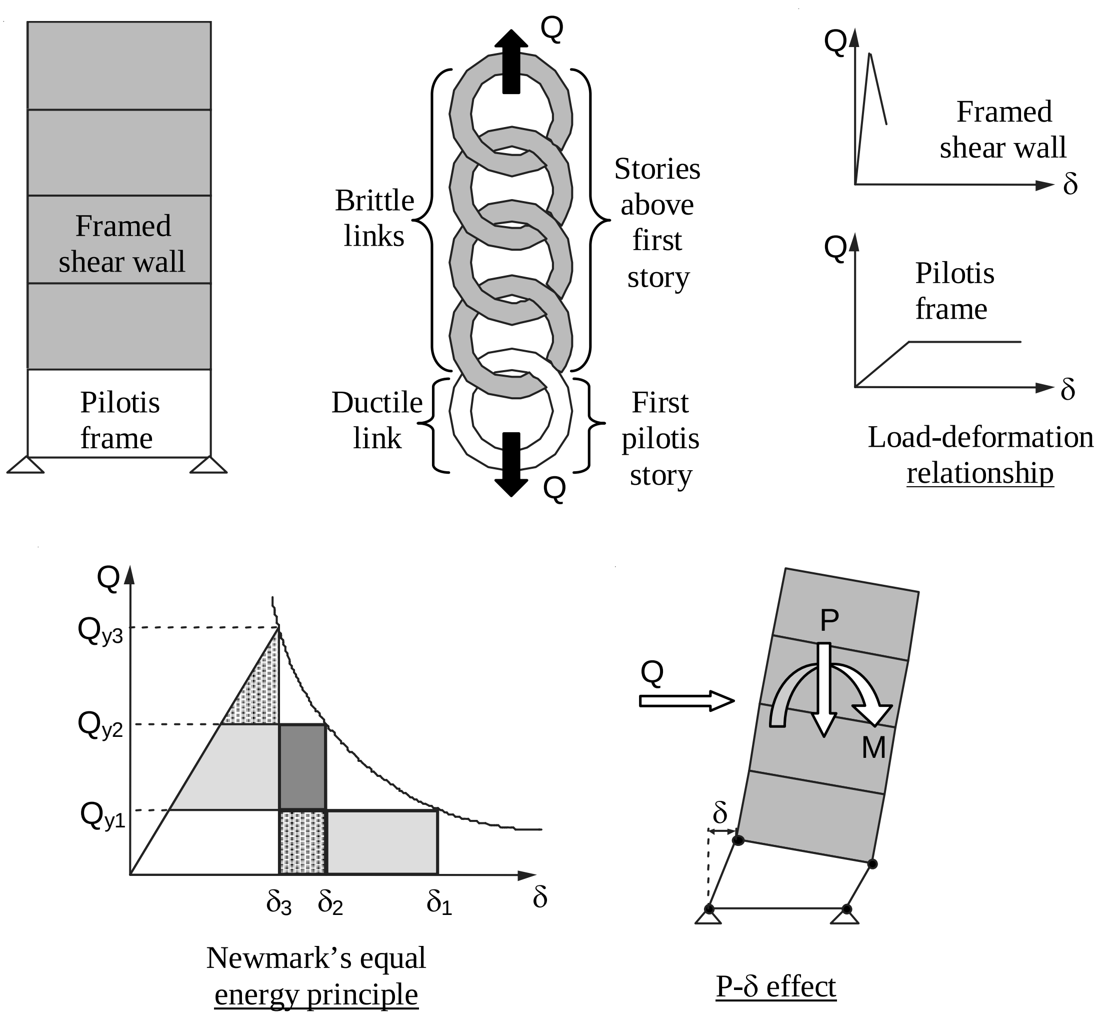



**Fig. 1 Mechanism of pilotis building**

column hinge mechanism, also referred to as a soft story, may impose large plastic hinge rotations, which even with good detailing of the affected regions, would be difficult to accommodate. In the past investigation (Yamakawa T. et al. 2001) by elastoplastic earthquake response analysis of six-story RC frame building with shear walls in all stories except in the first story, it was confirmed that by the enhancement of lateral strength of the soft first story, the story drift demand in that story can be reduced. Therefore, the overall deformation to be considered in the design will need to be limited to ensure that the maximum story drift angle at a critical locality do not become excessive.

From the past earthquake background, though it is identified that the seismic vulnerability in Okinawa is still lowest in Japan, but the 1999 Chi-Chi earthquake in Taiwan and the 2005 Fukuokaken Seiho-oki earthquake in Fukuoka, which are nearer to Okinawa, give the wakeup-call for the Okinawan people. However, the pilotis-type buildings although came to be recognized as a weak earthquake resistant structure, it is widely constructed in Okinawa. The factors in choice of pilotis buildings in

Okinawa are 1) scarcity of land as 19% of the Okinawa main island is occupied by the US military base; 2) a large number of private cars due to lack of mass transportation system; 3) the lowest zone coefficient (0.7) by Japanese code for seismic design load; 4) to provide good amenity through ventilation due to high temperature and high humidity.

Considering the detailed facts discussed above and based upon the past investigations of retrofitted RC wing-wall columns and bare frames (Rahman M. N. et al. 2004, 2005), a combination of strength-ductility-type and ductility-type retrofit techniques is proposed for first story pilotis frames. Nonlinear dynamic analyses using the "RUAUMOKO" software (Carr A.J. 1980-2007) are then carried out for a model representing a typical four story pilotis building in Okinawa to study the first story drift demand before and after retrofit applying the proposed retrofit technique.

# Chapter 2

## BACKGROUND OF PROPOSED RETROFIT TECHNIQUE

According to the proposed strength-ductility-type retrofit technique, the main square column is jacketed with channel-shaped steel plate and other steel plates are connected to this steel channel using PC bars to form a formwork of width equal to that of the column. The additional concrete is then cast to make a hybrid wall. After hardening of the post-cast concrete, initial tension forces are applied to PC bars that are previously penetrated across the wall. Cementing material is grouted to eliminate the gap between the column surface and the steel plate.

In the above retrofit technique, steel plates and PC bars can act as formwork and form-ties during the casting of the additional concrete. After hardening of the post-cast concrete, they can serve for shear strengthening as well as confinement, and can also maintain the rigidity and provide protection against the spalling of the cover concrete and the local buckling of longitudinal reinforcement. As columns and wing-walls are firmly united, they can act as unified members. As a result, shear strength is increased due to the formation of a large compression strut, and flexural strength is increased by the provision of a large, unified section with a large lever arm for moment resistance. Another significant point is that the thickness of the additional cast-in-site wall is the same as the width of the column, and thus the construction process is easy and effective confinement can be achieved. In this retrofit technique, essentially no flexural or shear reinforcement is provided inside the wall in either the longitudinal or the transverse direction. Therefore, the proposed retrofit technique is simple, convenient, economic, and effective than the conventional retrofit method. Because in the conventional method, additional reinforcements are provide in the additional wall along the longitudinal and transverse directions, and stud dowels are also used to connect the wall with the column. As a result, many holes are needed in the column and the beam, which is inconvenient as well as uneconomical. An example of conventional and proposed retrofit techniques of installation of infill wall into bare frame are illustrated in **Fig. 2.1**.

Again, according to the ductility-type retrofit only, the shear critical column is retrofitted by using corner blocks and PC bar prestressing like circumferential tie-hoop around the column. In this retrofit technique, PC bar prestressing serves for shear strengthening and confinement. Therefore, the shear failure of the column is prevented with ensuring ductility and the column can sustain vertical load.

The effectiveness of the proposed retrofit technique was verified through the cyclic loading tests of wing-wall column members as well as for one-story one-bay bare frames. The experimental results of wing-wall column and bare frame specimens are given in **Figs. 2.2.1** to **2.2.8**.

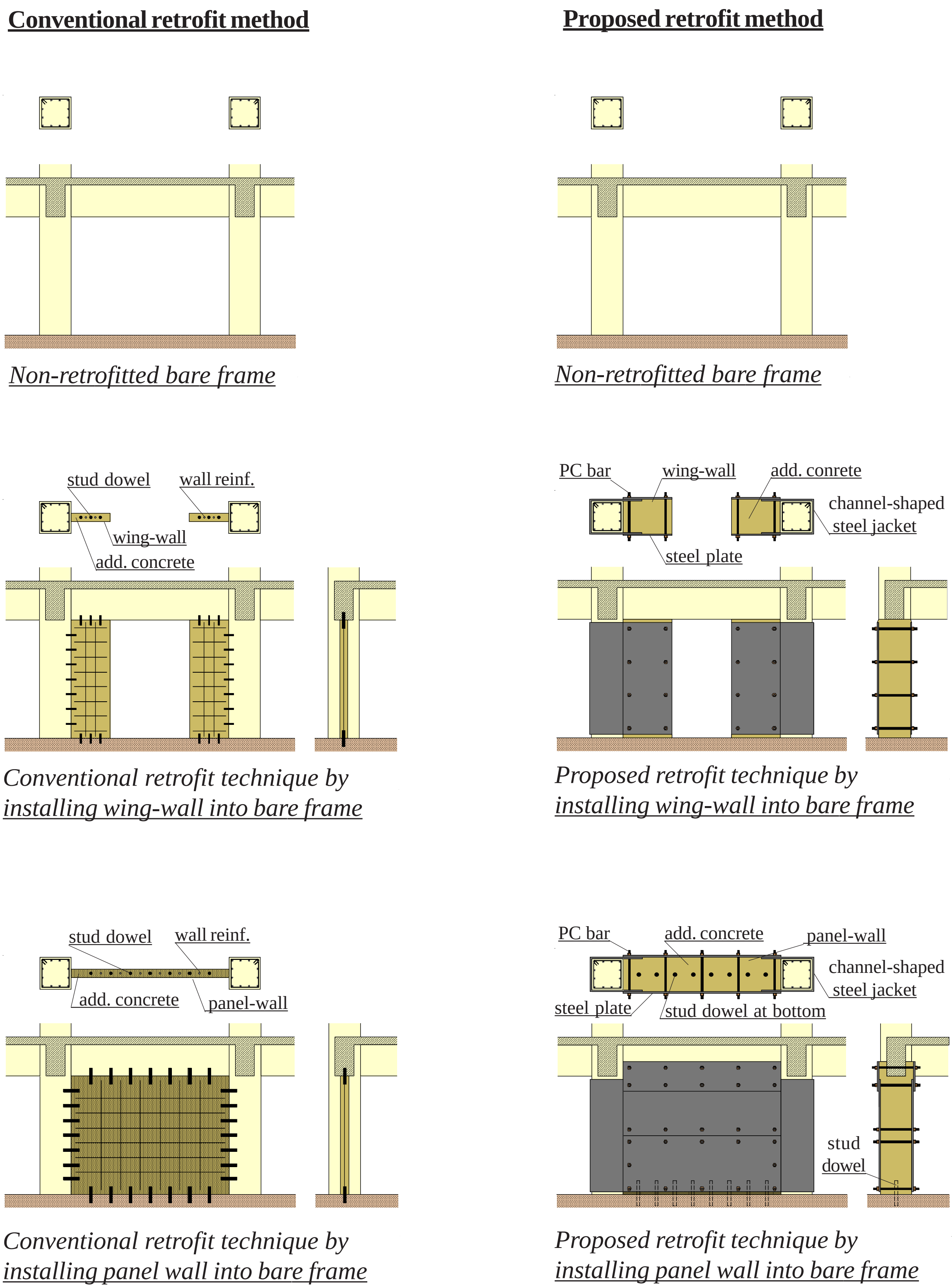
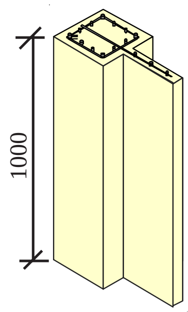
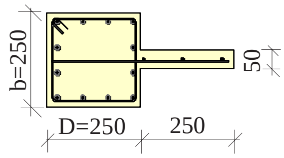



**Fig. 2.1 Example of conventional and proposed retrofit techniques**

**R03WO-P0**

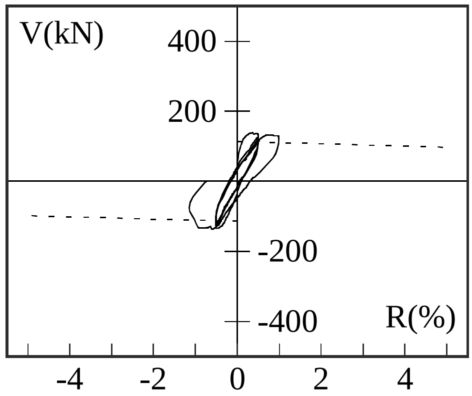



Elevation




Cross section ( unit:mm )

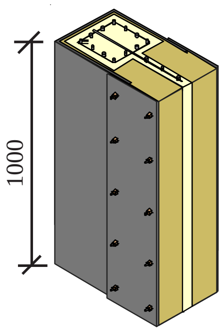
*Specimen details:*
Shear span to depth ratio, (M/(VD))=2.0 (column only); Axial force ratio, N/(bD$\sigma_B$) =0.2; Reinf. in column:- longitudinal reinf.: 12-D10 ($p_g$=1.36%), hoop: 3.7ϕ-@105 ($p_w$=0.08%); Reinf. in wall:- 3.7ϕ-@105 single (horizontal & vertical) ($p_w$=0.21%); Cylinder strength of concrete,$\sigma_B$=25.9 (MPa).



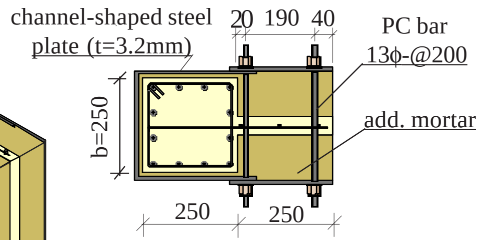

*Shear force - drift angle response*

**Fig. 2.2.1 Test result of specimen R03WO-P0**

**R03WO-S**

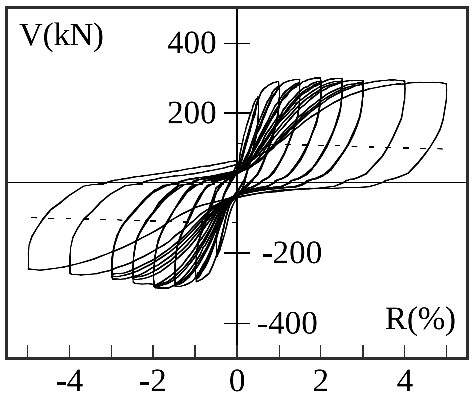



Elevation




Cross section ( unit:mm )

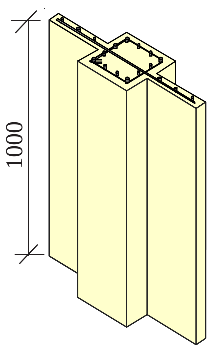
*Specimen details:*
Shear span to depth ratio, (M/(VD))=2.0 (column only); Axial force ratio, N/(bD$\sigma_B$) = 0.2; Reinf. in column:- longitudinal reinf.: 12-D10 ($p_g$=1.36%), hoop:3.7ϕ-@105 ($p_w$= 0.08%); Reinf. in wall:-3.7ϕ-@105 single ($p_w$=0.21%); Cylinder strength of concrete, $\sigma_B$=25.9 (MPa); Cylinder strength of add. mortar =68.5(MPa); Initial strain in PC bar= 1250μ.



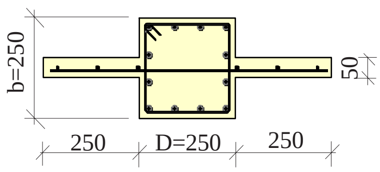

*Shear force - drift angle response*

**Fig. 2.2.2 Test result of specimen R03WO-S**

**R02WC-P0**

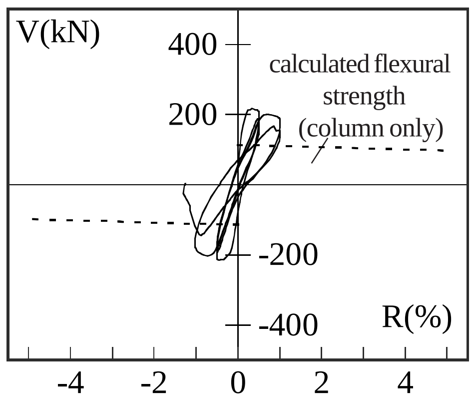



Elevation




Cross section ( unit:mm )

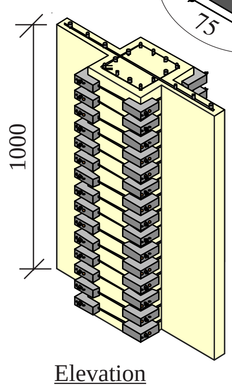
*Specimen details:*
Shear span to depth ratio, (M/(VD))=2.0 (column only); Axial force ratio, N/(bD$\sigma_B$) = 0.2; Reinf. in column:- longitudinal reinf.: 12-D10 ($p_g$=1.36%), hoop: 3.7ϕ-@105 ($p_w$=0.08%); Reinf. in wall:-3.7ϕ-@105 single (horizontal & vertical) ($p_w$= 0.21%); Cylinder strength of concrete,$\sigma_B$=25.7 (MPa).



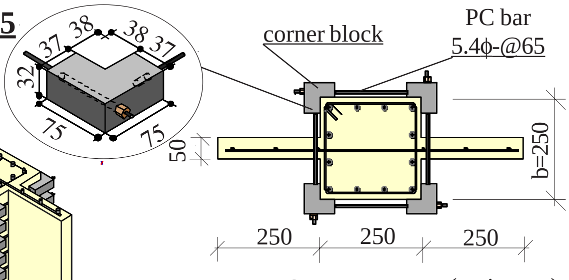

*Shear force - drift angle response*

**Fig. 2.2.3 Test result of specimen R02WC-P0**

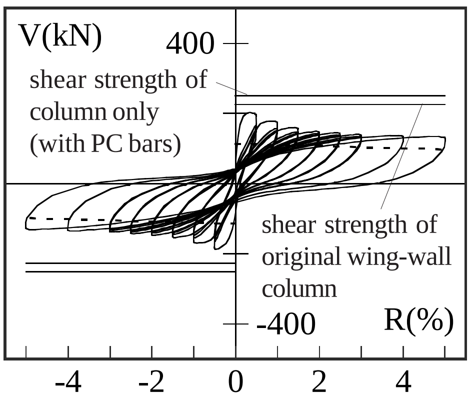
**R02WC-P65**




Elevation




Cross section ( unit:mm )

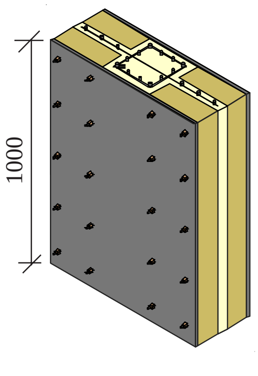



*Shear force - drift angle response*

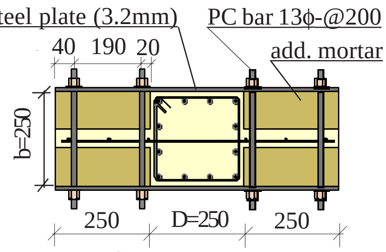
*Specimen details:*
Shear span to depth ratio, (M/(VD))=2.0 (column only); Axial force ratio, $N/(bD\sigma_B)$ = 0.2; Reinf. in column:- longitudinal reinf.: 12-D10 ($p_g$=1.36%), hoop: 3.7ϕ-@105 ($p_w$=0.08%); Reinf. in wall:- 3.7ϕ-@105 single (horizontal & vertical) ($p_w$=0.21%);
Cylinder strength of concrete, $\sigma_B$=25.7 (MPa); Initial strain in PC bar= 2450μ.

**Fig. 2.2.4 test result of specimen R02WC-P65**

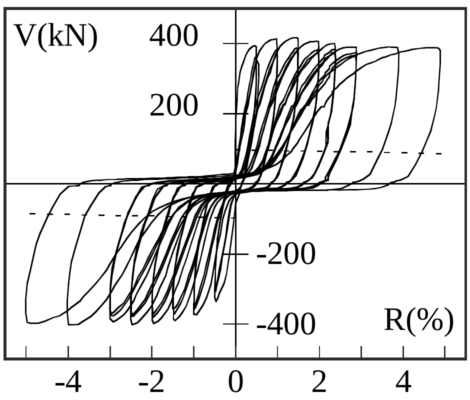
**R03WC-P200S**




Elevation




Cross section ( unit:mm )

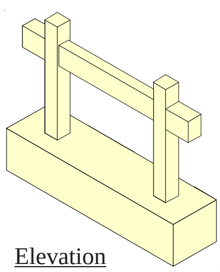



*Shear force - drift angle response*

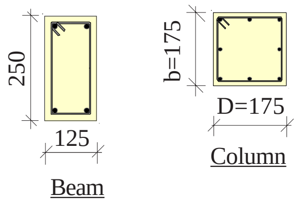
*Specimen details:*
Shear span to depth ratio, (M/(VD))=2.0 (column only); Axial force ratio, $N/(bD\sigma_B)$ = 0.2; Reinf. in column:- longitudinal reinf.: 12-D10 ($p_g$=1.36%), hoop: 3.7ϕ-@105 ($p_w$= 0.08%); Reinf. in wall:-3.7ϕ-@105 single ($p_w$=0.21%); Cylinder strength of concrete, $\sigma_B$=19.5(MPa); Cylinder strength of add. mortar =74.0 (MPa); Initial strain in PC bar= 1250μ.

**Fig. 2.2.5 Test result of specimen R03WC-P200S**

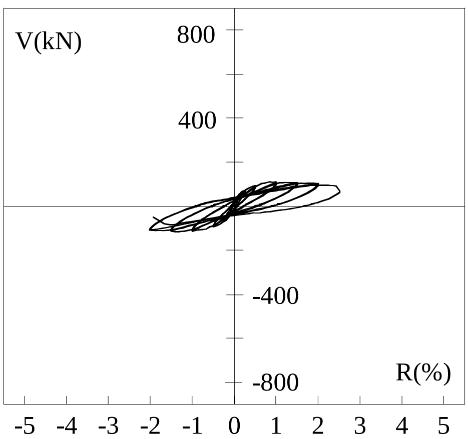
**R05P-P0**



Elevation




Cross section ( unit:mm )

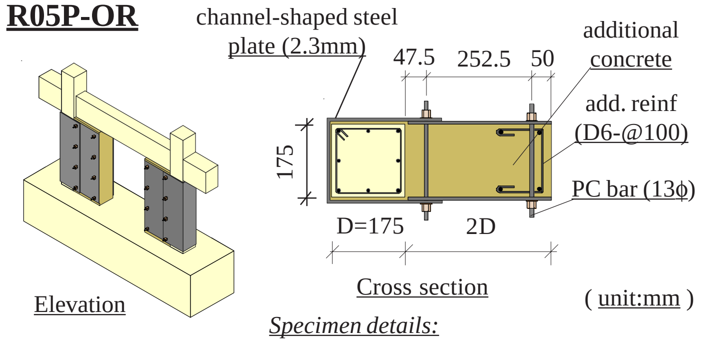



*Shear force - story drift angle response*

*Specimen details:*
Axial force ratio, $N/(bD\sigma_B)$ = 0.2 (per column); Reinf. in column:- longitudinal reinf.: 8-D10 ($p_g$=1.85%), hoop: 3.7ϕ-@105 ($p_w$=0.12%); Reinf. in beam:- longitudinal reinf.: 4-D13, (top and bottom) $p_g$= 1.63%), stirrup: D6-@120 ($p_w$=0.43%); Cylinder strength of frame concrete, $\sigma_B$=28.3 (MPa).

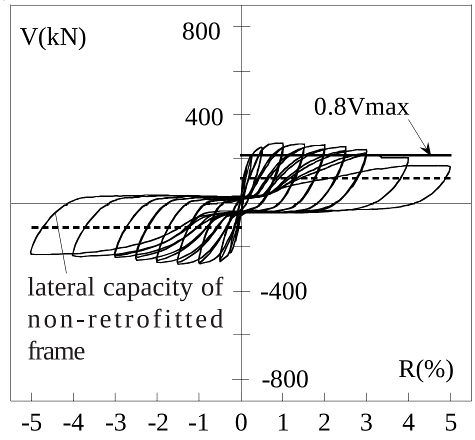
**Fig. 2.2.6 Test result of specimen R05P-P0**

**R05P-OR**




*Specimen details:*

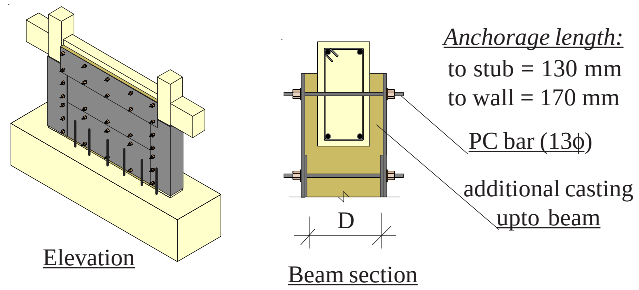
Axial force ratio, $N/(bD\sigma_B) = 0.2$ (per column); Reinf. in column:- longitudinal reinf.: 8-D10 ($p_g$=1.85%), hoop: 3.7ϕ-@105 ($p_w$=0.12%); Reinf. in beam:- longitudinal reinf.: 4-D13, (top and bottom) ($p_g$ = 1.63%), stirrup: D6-@120 ($p_w$=0.43%); Cylinder strength of frame concrete,$\sigma_B$=28.3 (MPa); Cylinder strength of additional concrete = 32.3 (MPa); Initial strain in PC bar at wall = 1250μ.

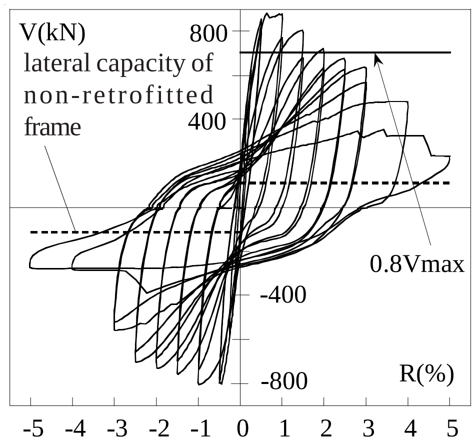



*Shear force - story drift angle response*

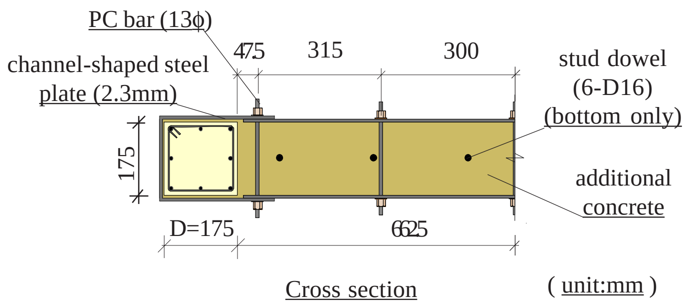
**Fig. 2.2.7 Test result of specimen R05P-OR**

**R05P-WDB**




*Shear force - story drift angle response*




*Specimen details:*

Axial force ratio, $N/(bD\sigma_B) = 0.2$ (per column);
Reinf. in column:- longitudinal reinf.: 8-D10 ($p_g$=1.85%), hoop: 3.7ϕ-@105 ($p_w$=0.12%);
Reinf. in beam:- longitudinal reinf.: 4-D13, (top and bottom) ($p_g$=1.63%),
stirrup: D6-@120 ($p_w$=0.43%);
Cylinder strength of frame concrete,$\sigma_B$=32.3 (MPa);
Cylinder strength of additional concrete =32.3 (MPa);
Initial strain in PC bar at wall = 1250μ.

**Fig. 2.2.8 Test result of specimen R05P-WDB**

# Chapter 3

# CALIBRATION OF HYSTERESIS RULES

## 3.1 Common flexural hysteresis rules

In order to obtain the nonlinear dynamic response of a member due to an earthquake excitation, it is necessary to model its nonlinearity with an appropriate hysteresis rule. There are numerous approaches for modeling inelastic behavior of the reinforced concrete structures (see **Fig. 3.1**). An important point in this task is the appropriate modeling of the inelasticity in the frame members. A hysteresis rule of a RC member describes the paths of loading, unloading and reloading at different states, such as before cracking, after cracking-before yielding, after yielding and etc. Many hysteresis rules are used for modeling the flexural behavior of reinforced concrete members, such as degrading bilinear model, Ramberg-Osgood model, Clough model, Modified Takeda rule and etc. (Otani S. 1981). The key point in selecting the hysteresis rule for a member is compatibility of the applied rule with its real behavior that their correlation can be clarified by comparison between obtained response of the hysteresis model and experimental result.

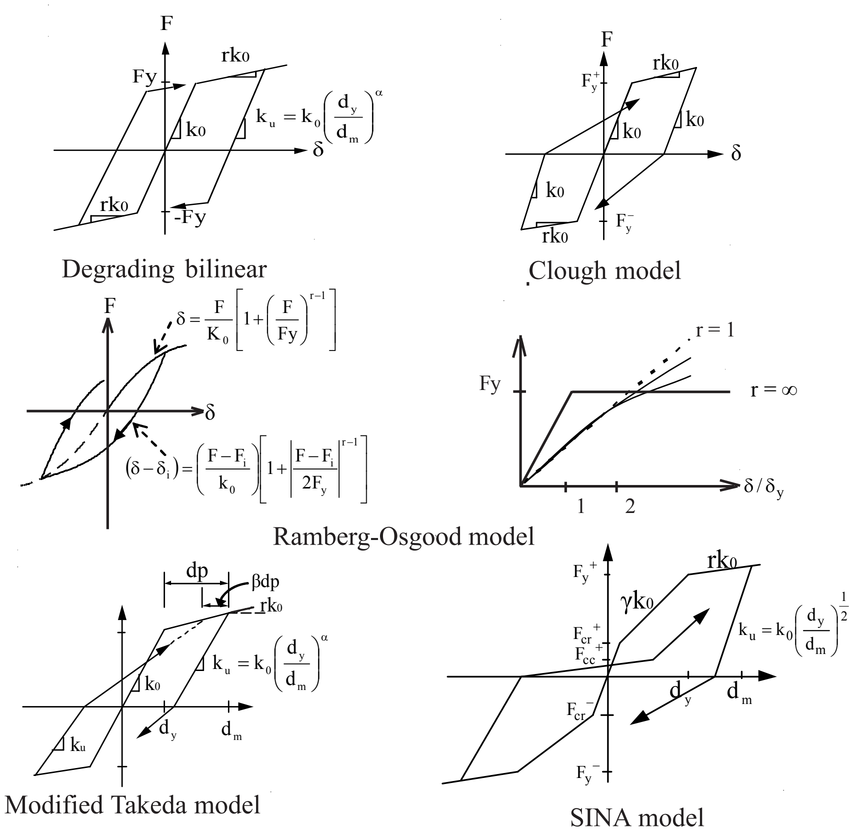



**Fig. 3.1 Hysteresis rules for modeling the flexural behavior**

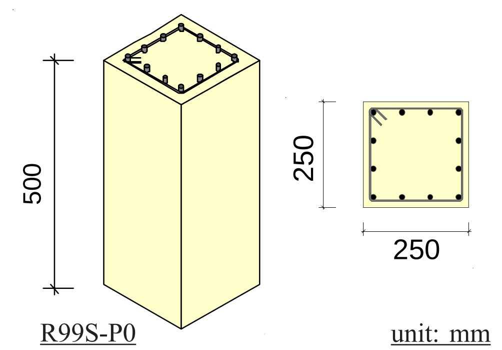
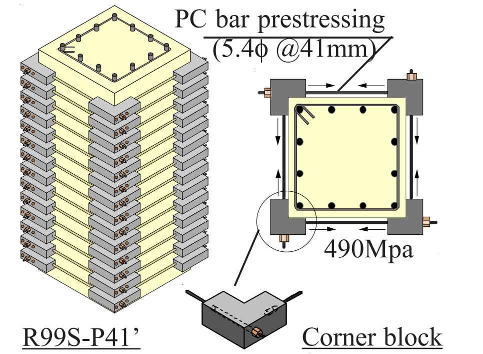
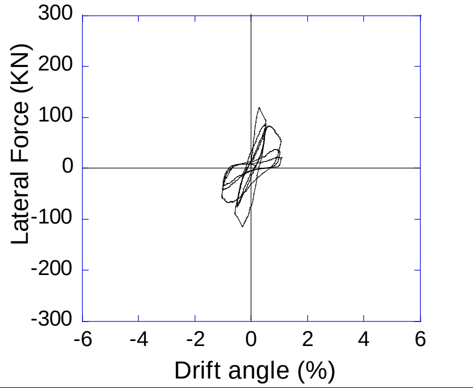
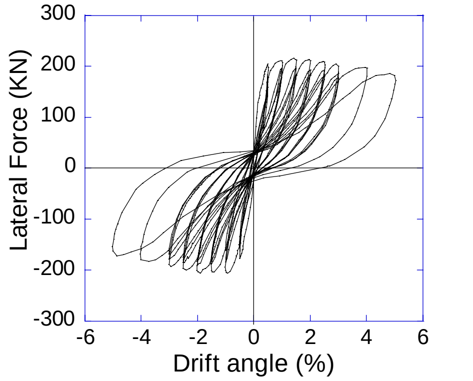
### 3.2 Calibration of hysteresis rule of the column retrofitted by PC bar prestressing

A new seismic retrofit technique utilizing PC bar prestressing was proposed as one of techniques in order to prevent shear failure and improve ductility of RC columns (Yamakawa T. et al. 2002). A series of investigation have been done to evaluate the lateral force capacity and to propose the design method of RC columns retrofitted by PC bar prestressing as external hoops on the base of experimental test results under the combination of cyclic lateral forces and a constant axial load whose level to concrete cylinder strength ratio is 0 2. These test results consist of total of 31 specimens whose failure modes are shear, flexural and bond failure ones. As a result, if seismic retrofit technique by PC bar prestressing is designed so that shear and bond strength of the retrofitted columns can overcome their flexural strength, a desirable seismic performance may be expected for seismic retrofit of RC columns. The test setup and loading program are illustrated in **Fig. 3.2**.

In order to model column retrofitted by PC bar prestressing, one specimen is used to assess and calibrate the hysteresis rule for dynamic analysis (see **Table 3.1**). Since in this calibration procedure the assignment of the hysteresis rule is based on the experimental result, the obtained behavior of the member during the test should be verified to select the appropriate hysteresis rule. In **Table 3.1**, the experimental result shows that the retrofitted column has flexural behavior where the pinching effect is significant. Also, after crack the stiffness of the member markedly reduces that this behavior can significantly affect the dynamic response of the member. Among the flexural hysteresis rules that are shown **in Fig. 3.1**, it seems that SINA hysteresis rule is more suitable to model the mentioned behavior. The SINA hysteresis rule is a trilinear hysteresis rule to model stiffness degradation of reinforced concrete members in flexure (Car A. J. 1980-2007). As shown in **Fig. 3.3**, the key parameters to define modified SINA hysteresis rule are; bilinear factor(r), unloading power factor (á), pinching factor(d1/d0), cracking action as ratio of yield strength (FCR) and cracking closing as a ratio of yield strength(FCC). For achieving the best agreement between the analytical and experimental results, some modifications in original model are done. These modifications consist in changing the points of crack and crack closing forces as a ratio of yield strength and defining the deformation coordinate of pinching pointd ($d_0$) as a ratio of residual deformation in first hysteretic cycle ($d_1$). The definition of deformation coordinate controls the pinching point and represses overshooting during the dynamic analysis.

The specimen is modeled as a spring element in two-dimension computer program RUAUMOKO (Carr A. J. 1980-2007). This element has two independent springs in vertical and horizontal directions in which horizontal one describes the nonlinear lateral behavior of the member (see **Fig. 3.4**). After modeling the specimen and analyzing the model under displacement control procedure, the obtained hysteresis response shows that it agrees well with the analytical result

**Table 3.1 Comparision between original column and retrofitted one by PC bar**

| | Original column | Retrofitted column by PC bar prestressing |
|---|---|---|
| |  |  |
| Specimen details | Shear span to depth ratio, (M/(VD))=2.0, Axial force ratio, $N/(bD\sigma_B)$ =0.2; Reinf. in column:- longitudinal reinf.:12-D10 ($p_g$=1.36%), hoop: 3.7ϕ-@105 ($p_w$=0.08%), Yeild strength of steel rebar: 371(MPa), Yeild strength of hoop: 333(MPa) Cylinder strength of concrete, $\sigma_B$=19.5 (MPa). | Shear span to depth ratio, (M/(VD))=2.0, Axial force ratio, $N/(bD\sigma_B)$ =0.2; Reinf. in column:- longitudinal reinf.:12-D10 ($p_g$=1.36%), hoop: 3.7ϕ-@105 ($p_w$=0.08%), Yeild strength of steel rebar: 371(MPa), Yeild strength of hoops: 333(MPa), Cylinder strength of concrete $\sigma_B$=19.5 (MPa), PC bar 5.4ϕ−@41, Yeild strength of PC bar: 1202(MPa) |
| Experimental result |  |  |

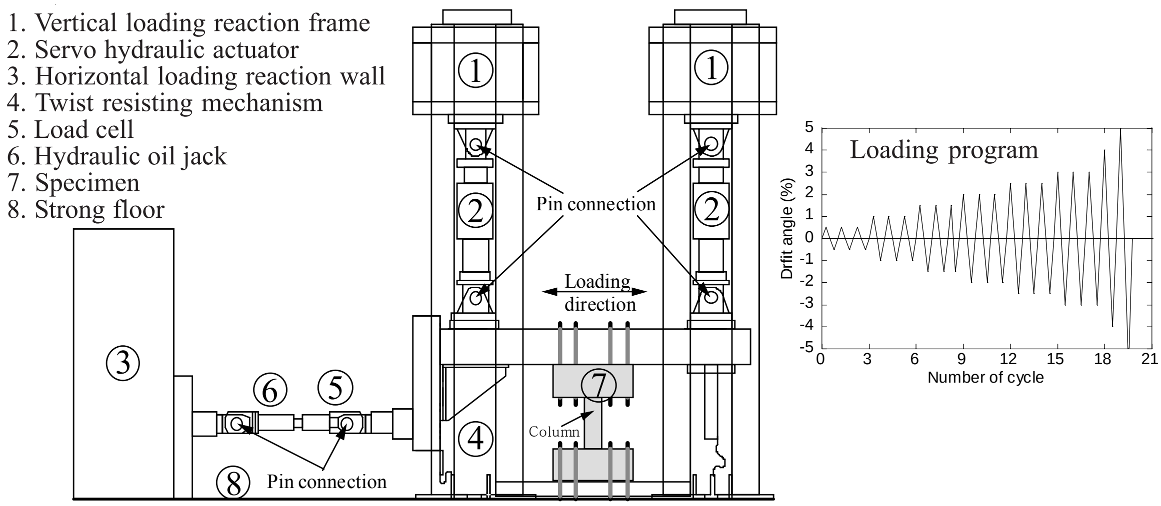



**Fig. 3.2 Test setup and loading program**

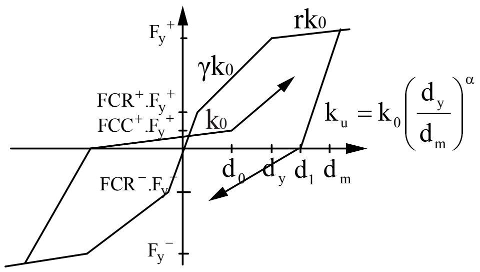



**Fig. 3.3 Modified SINA model**

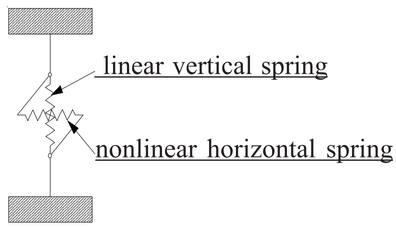



**Fig. 3.4 Simulated specimen**

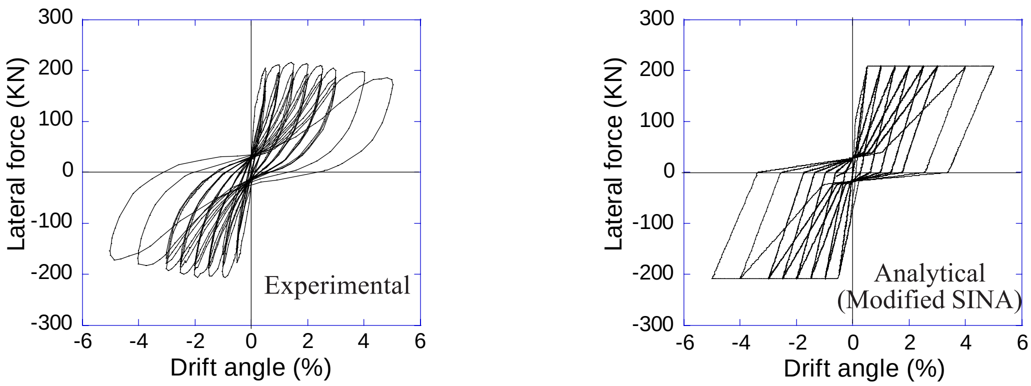


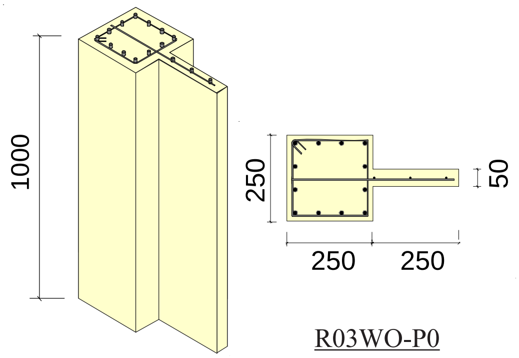
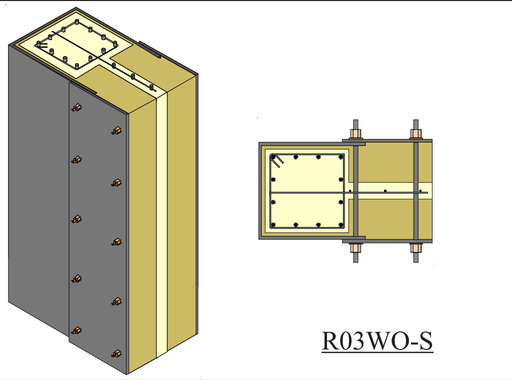
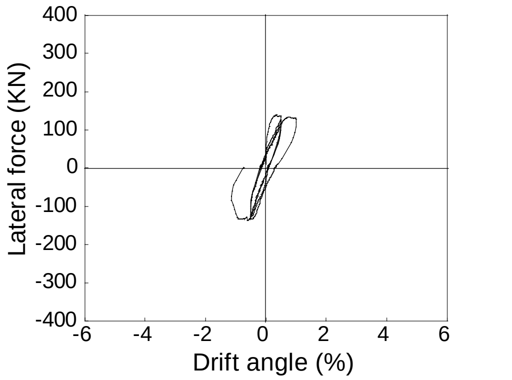
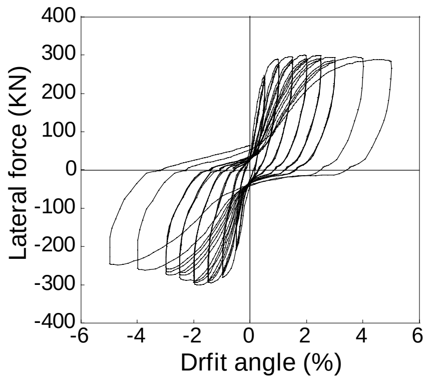

**Fig. 3.5 Comparison of test and analytical results of column retrofitted by PC bar prestressing**

that computed by modified SINA hysteresis rule (see **Fig. 3.5**). In the experimental hysteresis behavior, it is obvious that the strength at large drift angle degrades, so, after dynamic analysis it can be verified that the drift angle response falls in this range or not.

### 3.3 Calibration of hysteresis rule of the column retrofitted by thick hybrid wall

One of the method that effectively enhance the strength, stiffness and ductility of columns is utilizing of thick hybrid wall. Details of application of this technique and calibration of its hysteresis rule are explain here for a test specimen (Yamakawa T. etal. 2006). As shown in **Table 3.2**, the main part of this specimen contains column and wing wall. The main square column was jacketed with channel-shaped steel plate and then additional steel plates (thickness = 2.3 mm) were connected with this plate by utilizing PC bars (diameter = 13mm) to form a formwork. Additional mortar was cast within formwork. After hardening of the post-cast mortar and prior to loading test, initial tension force with a strain of about 1,250μ was applied in PC bars that were inserted across the wing-wall beforehand. The epoxy resin was grouted to fill the gap within the column surface and channel-shaped steel plate. The test setup and loading program are the same as used for column retrofitted by PC bar prestressing (see **Fig. 3.2**). The computer modeling of this specimen is identical to that of the column retrofitted by PC bar prestressing. For analytical simulation, the retrofitted wing-wall column is simulated by spring element and the modified SINA rule is assigned. As shown in **Fig. 3.6**, the comparison between analytical and experimental results confirms that the applied hysteresis rule defines well the real behavior of the wing-wall column retrofitted by thick hybrid wall.

**Table 3.2 Non-retrofitted and retrofitted wing wall column**

| | Original wing-wall column | Retrofitted wing-wall column |
|---|---|---|
| |  |  |
| Specimen details | Shear span to depth ratio, (M/(VD))=2.0 (column only); Axial force ratio, N/(bD$\sigma_B$) =0.2; Reinf. in column:- longitudinal reinf.: 12-D10 ($p_g$=1.36%), hoop: 3.7ϕ-@105 ($p_w$=0.08%); Reinf. in wall:- 3.7ϕ-@105 single (horizontal & vertical) ($p_w$=0.21%); Cylinder strength of concrete,$\sigma_B$=25.9 (MPa). | Shear span to depth ratio, (M/(VD))=2.0 (column only); Axial force ratio, N/(bD$\sigma_B$) = 0.2; Reinf. in column:- longitudinal reinf.: 12-D10 ($p_g$=1.36%), hoop:3.7ϕ-@105 ($p_w$= 0.08%); Reinf. in wall:-3.7ϕ-@105 single ($p_w$=0.21%); Cylinder strength of concrete, $\sigma_B$=25.9 (MPa); Cylinder strength of add. mortar =68.5(MPa); Initial strain in PC bar= 1250μ. |
| Experimental result |  |  |

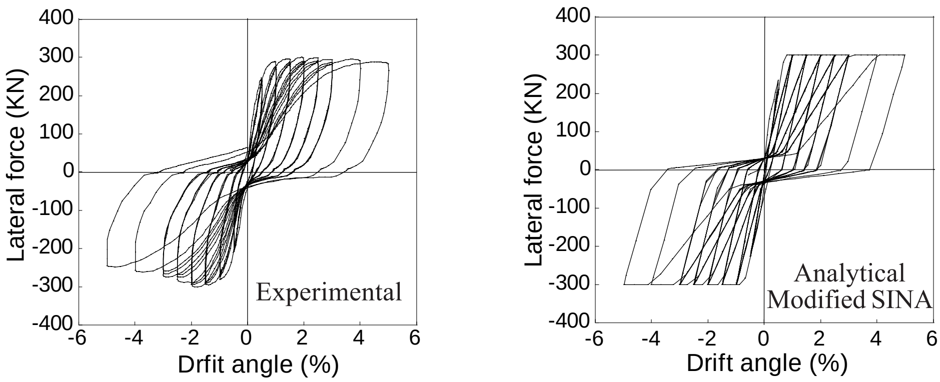



**Fig. 3.6 Comparison of test and analytical results of wing-wall column retrofitted by thick hybrid wall**

# Chapter 4

# DYNAMIC ANALYSIS OF AN EXISTING PILOTIS BUILDING

## 4.1 Details of an existing pilotis building

To verify dynamic performance of pilotis-type building, an existing pilotis building in Okinawa Island is selected. The building has a rectangular plan that the lateral stiffness of the building in two directions is different. In retrofit procedure, the critical direction with less lateral stiffness is selected to encounter critical condition. The elevations of the building and the first story plan in the considered direction are illustrated in **Fig. 4.1**. The details of columns and beams are shown in **Table 4.1** and **Table 4.2**, respectively.

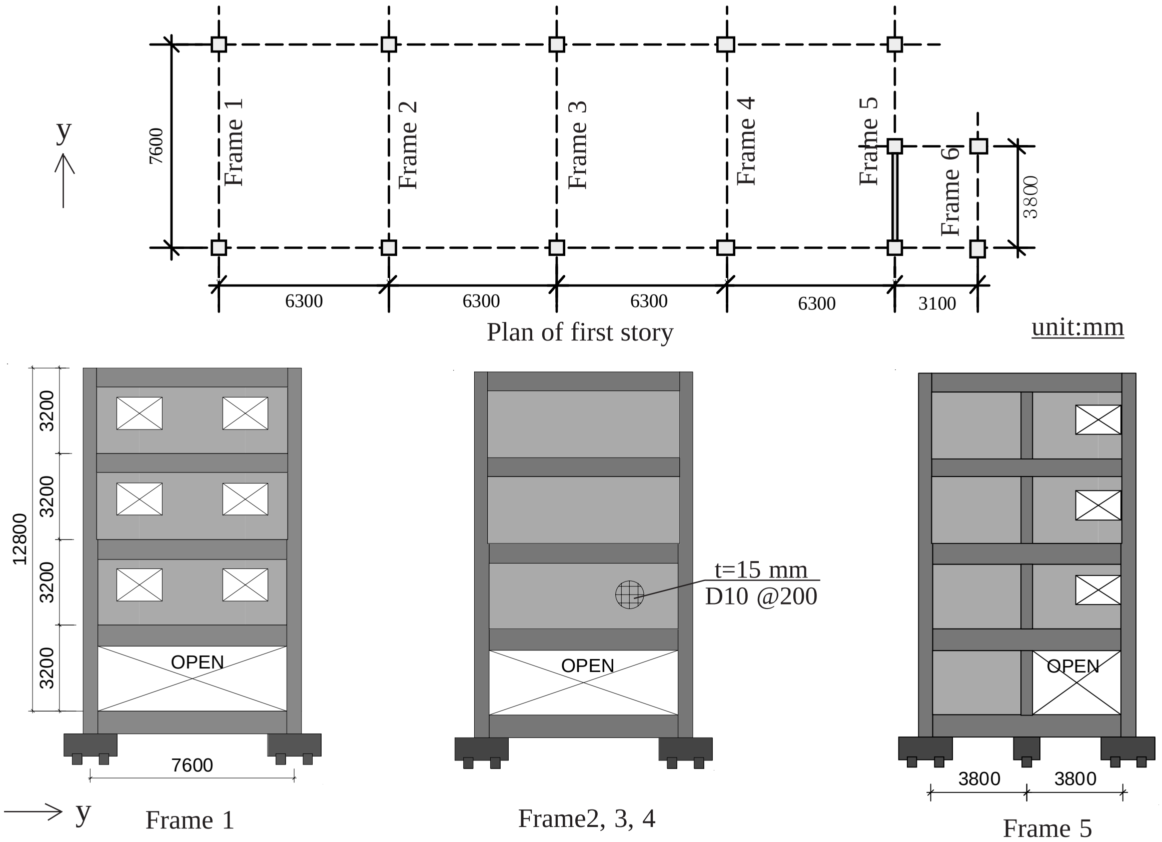



**Fig. 4.1 First story paln and elevations of an existing pilotis building**

**Table 4.1 Details of columns**

| Details ↑y | B D | B D |
|---|---|---|
| Level | Story 1, 2 | Story 3, 4 |
| BxD | 550x550 | 500x500 |
| Rebar | 14-D22 | 12-D22 |
| Hoop | D10@100 | D10@100 |

**Table 4.2 Details of beams**

| Details | B D | B D | B D |
|---|---|---|---|
| Level | Footing | All stories | All stories |
| Position | continuous | ends | middle |
| BxD | 400x850 | 350x700 | 350x700 |
| Top rebar | 5-D22 | 8-D22 | 2-D22 |
| Middle rebar | - | - | 2-D22 |
| Bottom rebar | 3-D22 | 4-D22 | 5-D22 |
| Hoop | D10@150 | D13@100 | D13@100 |

## 4.2 Modeling of the pilotis building

The mentioned pilotis building is modeled in computer program RUAUMOKO for nonlinear dynamic analysis. As discussed in preceding section, only the frames of critical direction are modeled. As shown in **Fig. 4.2**, to verify the global responses of the building, it is modeled as five frames that were laid in series and connected with rigid links at each story level. In this model, for simplicity frame 6 is not considered, but half of the weight of floor between frame 5 and 6 is applied on frame 5. The main components of structure in the modeling include columns, beams and shear walls. In this idealized model, the beams and floor systems are assumed to be rigid in flexure. The main components, columns and shear walls, were simulated as beam-column and spring element, respectively. The beam-column member is defined as an element that axial force in the member affects the current yield moment at each end. This element consists of two rigid links at both ends and a flexible length in the middle (see **Fig. 4.2**). The rigid end-blocks may incorporate within the length of any of the frame members, and in this case they are used to model the rigid length of member at the joint region. Also, the spring element may be used to simulate independent behavior of member in each direction. This element has three components that any of them can be defined by linear or nonlinear behavior. In this case, the shear wall is modeled as a spring element to simulate the transverse stiffness.

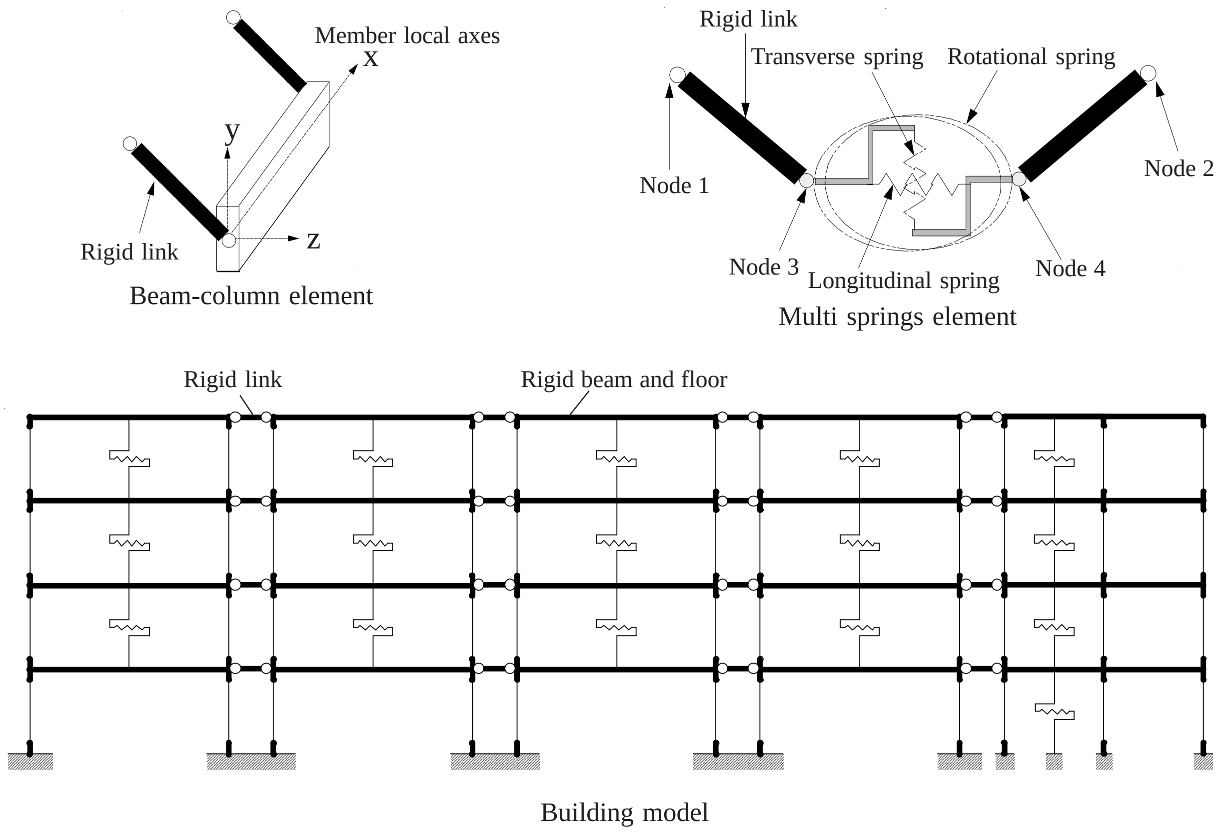



**Fig. 4.2 Modeling of the pilotis building**

## 4.3 Modal analysis of the pilotis building

Modal analysis is carried out in the computer program, RUAUMOKO, even though the results are not generally used during dynamic analysis. A modal analysis does provide a check on the structural data in that the natural periods and mode shapes are what would be expected for such a structure. In some cases a modal analysis is necessary as the natural frequency of the free vibration, and in some cases the mode shapes, are used to generate the appropriate damping for the structure. In this case the modal analysis is carried out to verify the influence of significant difference in lateral stiffness of first story relative to upper stories on the mode shapes. The number of modes required in a modal analysis is specified in many codes to be such that the sum of the effective weight of the mode used must be at least of the order of 90% of the total weight of the structure. The mode shapes of the pilotis building are shown in **Fig. 4.3**. Because of abrupt change in lateral stiffness of the building in the first story, the first mode demonstrates a concentrated displacement in the first story level. The modal analysis shows that the participation factor of the first mode is significant and so, it is rational to call it fundamental mode (see **Table 4.3**). It is expected that the global responses of the building will be similar to the fundamental mode.

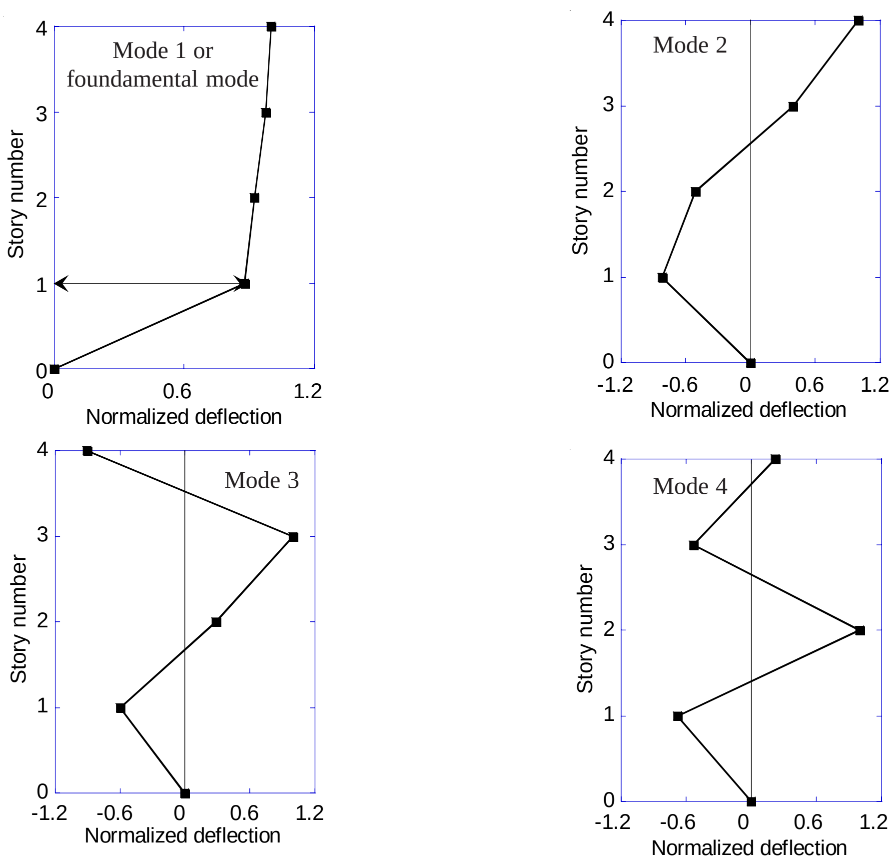



**Fig. 4.3 Mode shapes of the pilotis building**

**Table 4.3 Results of modal analysis**

| | Mode 1 | Mode 2 | Mode 3 | Mode 4 |
|---|---|---|---|---|
| Period (sec) | 2.408 | 0.0472 | 0.0252 | 0.0196 |
| Participation factor | 1.059 | -0.068 | -0.013 | -0.010 |
| Effective mass(%) | 100 | 0 | 0 | 0 |

## 4.4 Input earthquake waves

For dynamic analysis of the pilotis building, three earthquakes (namely, El Centro, Hachinohe, Taft) are used. The evaluation of the building is based on the maximum responses due to these earthquake records.According to Japanese guideline, the record should scaled according to the maximum velocity of 50 cm/s. Because the building is located in Okinawa island, the base shear coefficient is reduced to 0.7. On the other hand, the original records should be scaled according to the maximum velocity of 35 cm/s. The specification of the original and scaled records are shown in **Table 4.4**.

**Table. 4.4 Intensity of original and scaled earthquake records**

| Earthquake input wave | Original record | | Scaled record | |
|---|---|---|---|---|
| | PGA(cm/s$^2$) | PGV(cm/s) | PGA(cm/s$^2$) | PGV(cm/s) |
| El Centro NS | 341.7 | 33.5 | 357.5 | 35.0 |
| Taft EW | 175.9 | 17.7 | 347.7 | 35.0 |
| Hachinohe EW | 182.9 | 35.8 | 178.8 | 35.0 |

## 4.5 Dynamic responses of the pilotis building before retrofitting

Before dynamic analysis, the hysteresis rule of members should be defined, so it is essential to know the behavior of the members. On the other hand, it should be recognized that members fail in shear or flexure, and then according to obtained mechanism the compatible hysteresis rules will be defined. Because the flexural capacity of section depends on its axial force, and the axial force varies during the analysis, the calculation of flexural capacity needs a primary dynamic analysis to find the yield moment derived from interaction axial force-moment curve. At first, it is assumed that the columns and the shear walls fail in flexure and shear, respectively. So, the modified Takeda rule is defined for the columns, and origin centred rule for the shear walls. After the first analysis, it was observed that yield moments of columns were close to their calculated shear strength according to the JBDPA guidelines (JBDPA 2001). For this reason, the origin centered rule is assigned instead of modified Takeda model to represent the brittle behavior of the

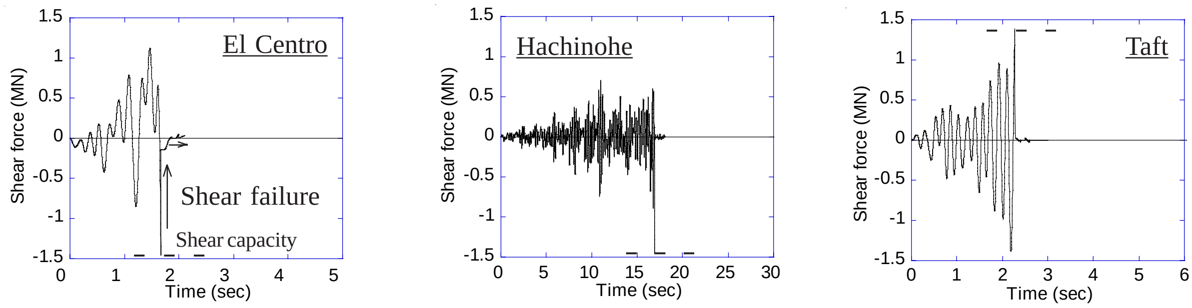


**Fig. 4.4 Response of the first story shear wall at frame 5 before retrofit**

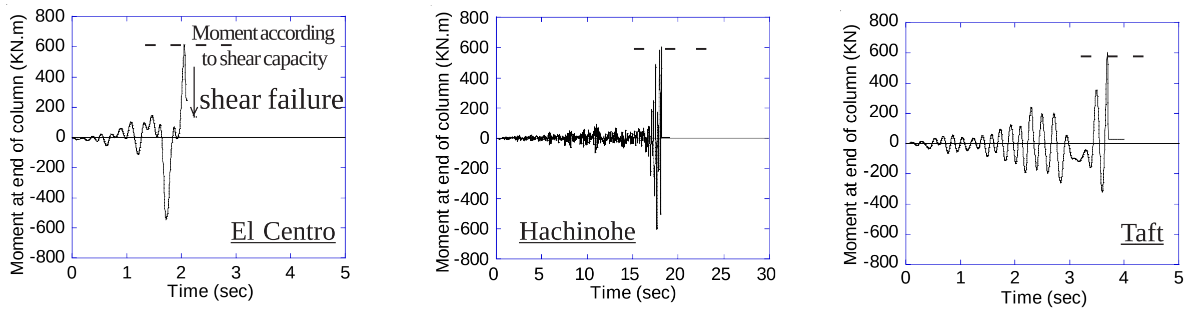


**Fig. 4.5 Response of the first story column at frame 2 before retrofit**

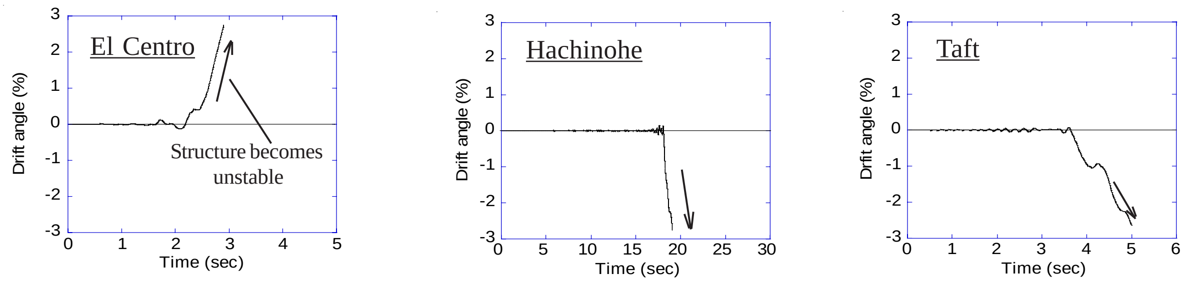


**Fig. 4.6 Drift angle response of the first story before retrofit**

columns. Strength degradation pattern is also used to take into account the sharp reduction in strength due to the shear failure. During the analysis, it is detected that at first, the shear wall in the first story of the frame 5 fail in shear and then the columns of the first story. As shown in **Fig. 4.4**, for three earthquakes after a few seconds the shear wall fail. Following the failure of the shear wall, as it is shown in **Fig. 4.5**, the columns fail in shear and the delay times depend on the input wave. Certainly, when the main members (namely the shear wall and the columns) fail, three is no lateral resistance force to resist the applied force that lead to collapse of the first story (see **Fig. 4.6**).

## 4.6 Dynamic analysis of the pilotis building retrofitted by ductility type method

After analyzing the existing pilotis building, it is revealed that the columns of the first story have brittle behavior in that the shear failure happened. So, the first strategy is the prevention of the shear failure in the columns that can be achieved by application of ductility type retrofit.

Utilizing of PC bar prestressing can be effective to improve the ductility of the first story columns, whereas it is uncertain that the lateral strength of the first story is sufficient or not. As discussed in chapter 3, this technique increase the shear strength of the columns and converts the shear failure mechanism to flexural one. Therefore, for the prevention of shear failure, all the columns in the first story are retrofitted by PC bar prestressing. The details of the retrofitted column are illustrated in **Fig. 4.7**. The column retrofitted by PC bar prestressing is modeled as spring element that a horizontal spring at the middle of the member is assigned by SINA hysteresis rule to represent the nonlinearity in this direction. After analysis , the results show that the drift angle of the first story is out of the allowable range with maximum value of 1% (see **Fig. 4.8(a)**). Therefore, the lateral strength of the first story should be increased to limit the drift angle response of the first story to allowable range. Also, in order to verify the influence of hysteresis rule on the responses of the building, the columns retrofitted by PC bar prestressing are again modeled by modified Takeda rule which is commonly used to model the flexural behavior of concrete members. Because the lateral deflection of the building is concentrated in the first story and the columns of the first story are only retrofitted, so, the first story response is verified to compare the responses due to SINA and Takeda hysteresis models. As shown in **Fig. 4.8(b)** , there is difference between the responses in two cases, and this difference is more significant for large deformation. This investigation proves that the maximum response depends not only on the yield strength of the members but also on the defined hysteresis rule. Specially, in this case it is seemed that the three linear behavior of the SINA model and the consideration of pinching effect affect the maximum responses and residual deflections, irrespectively. In **Fig. 4.9**, the hysteresis behavior of the first story due to El Centro earthquake for two cases are shown. The residual deflection of the first story depends on the assigned hysteresis rule in that the residual deflection for Takeda model is 4 times of SINA model due to El Centro earthquake(see **Fig. 4.10**). From the above discussion, it is concluded that the applied retrofit strategy should be changed in that not only the lateral ductility of the columns improve but it is also necessary to increase the strength and stiffness of the columns to limit the first story deflection within the allowable range.

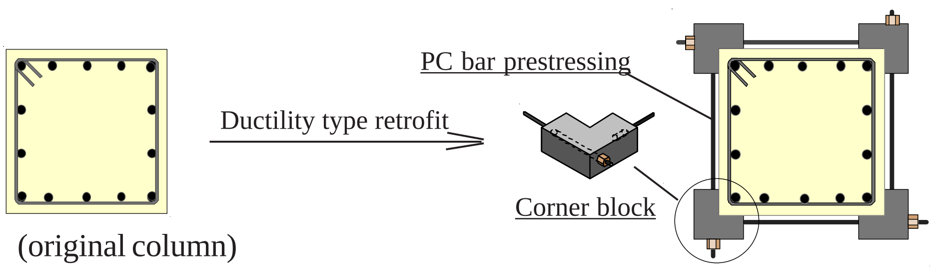



**Fig. 4.7 Retrofiting column by PC bar prestressing**

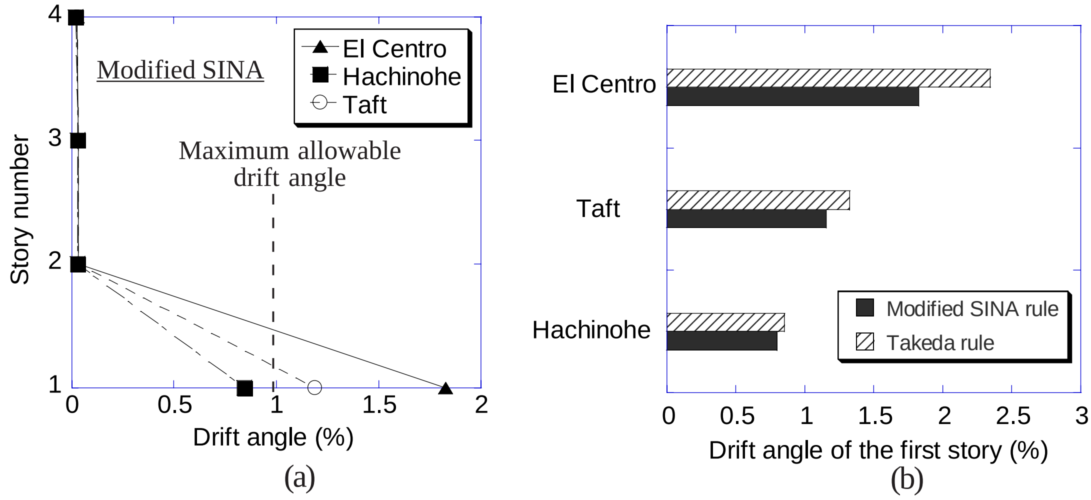



**Fig. 4.8 (a) Maximum drift angle of the stories (b) Comparision of the first story response between modified SINA and Takeda rules.(ductility-type retrofit)**

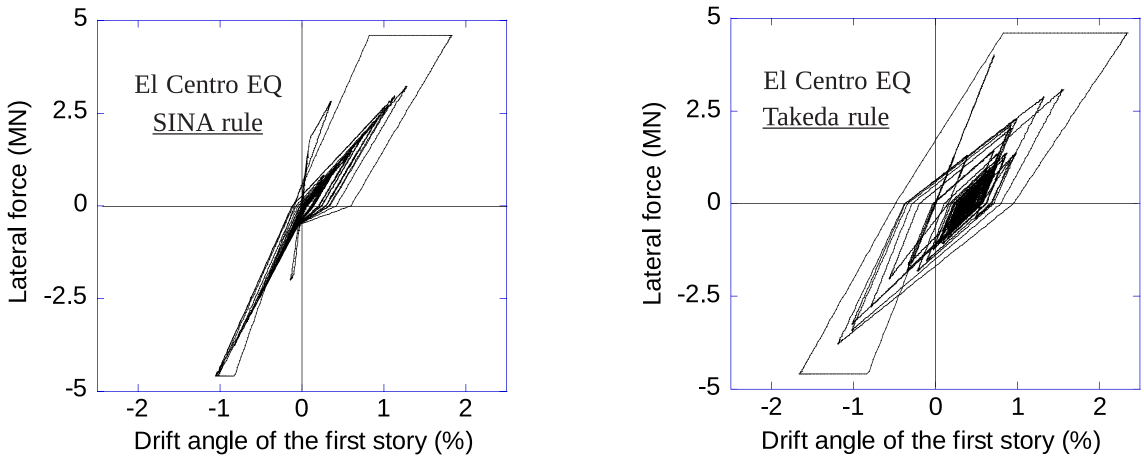



**Fig. 4.9 Comparison of hysteresis respose of the first story in ductility-type technique**

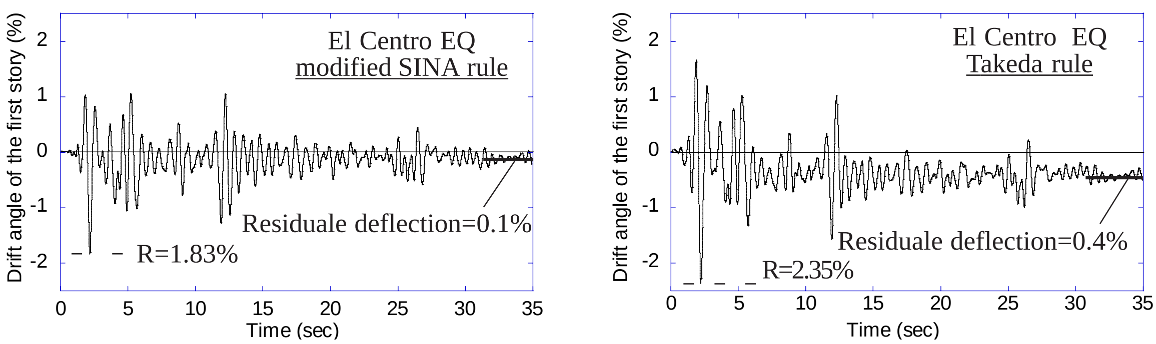



**Fig. 4.10 Comparison of first story response in ductility-type retrofit**

### 4.7 Dynamic analysis of the pilotis building retrofitted by strength-ductility type technique

The analysis of the pilotis building showed that the columns of the first story have brittle behavior, so ductility-type retrofit is essential to improve their ductility. After retrofitting the building by ductility-type technique (PC bar prestressing), the result demonstarted that the lateral displacement of the first story exceeded the maximum allowable drift angle. So, the application of ductility type retrofit only is not sufficient in this case. Employment of strength-ductility-type retrofit can enhance both the strength and the ductility of the system. Utilizing thick hybrid wall , one of the strength-ductility type retrofit techniques, effectively enhances the strength and the ductility of columns. Because all columns of the first story have brittle behavior, the ductility type retrofit of all columns is necessary. Besides, if the strength of all columns increase, the total lateral strength of the first story enhances, and maybe led to shear failure of upper stories. So, it is reasonable to use combination of ductility type and ductility-strength type retrofit in that the shear failure of the columns are prevented and the lateral displacement of the first story falls in the allowable range. The proposed retrofit method contains application of PC bar prestressing and thick hybrid wall in where frames 2, 3, 4 are rertrofitted by thick hybrid wall and rest columns retrofitted by PC bar prestressing. The details of retrofit plan is shown in **Fig. 4.11**. As it is shown in **Fig. 4.12(a),** although the deformation of the first story is larger than upper stories, it locates in allowable range. So, these result proves that employment of combination of thick hybrid wall and PC bar not only improves the ductility of the first story columns but also increase the lateral strength and stiffness of the first story that led to a acceptable performance of the retrofitted building. Another noticeable point in the retrofit procedure is influence of retrofitting of the first story on the upper story, and specially the second story. For this reason the shear force produced in the stories are obtained to compare with their capacity. The results show that the shear response of the stories do not exceed their shear capacity ,so the superstructure will be safe.

### 4.8 Discussion about retrofit procedure

In order to follow the retrofitting procedure from beginning to end, the results of the building before retrofit, after utilizing ductility type retrofit (PC bar prestressing) and after employment of strength-ductility-type retrofit (combination of PC bar and thick hybrid wall) are compared. As shown in **Fig. 4.13** and **Fig. 4.14**, before retrofit the drift angle response of the first story goes to infinity. This physically means that the first story completely collapses. After utilizing ductility type retrofit, the shear failure of the first story columns are prevented, but the lateral deformation exceeds the maximum allowable value, so this plan fails. At last, employment of strength-ductility type retrofit solves the problems by improving the strength, ductility, and stiffness of the first story.

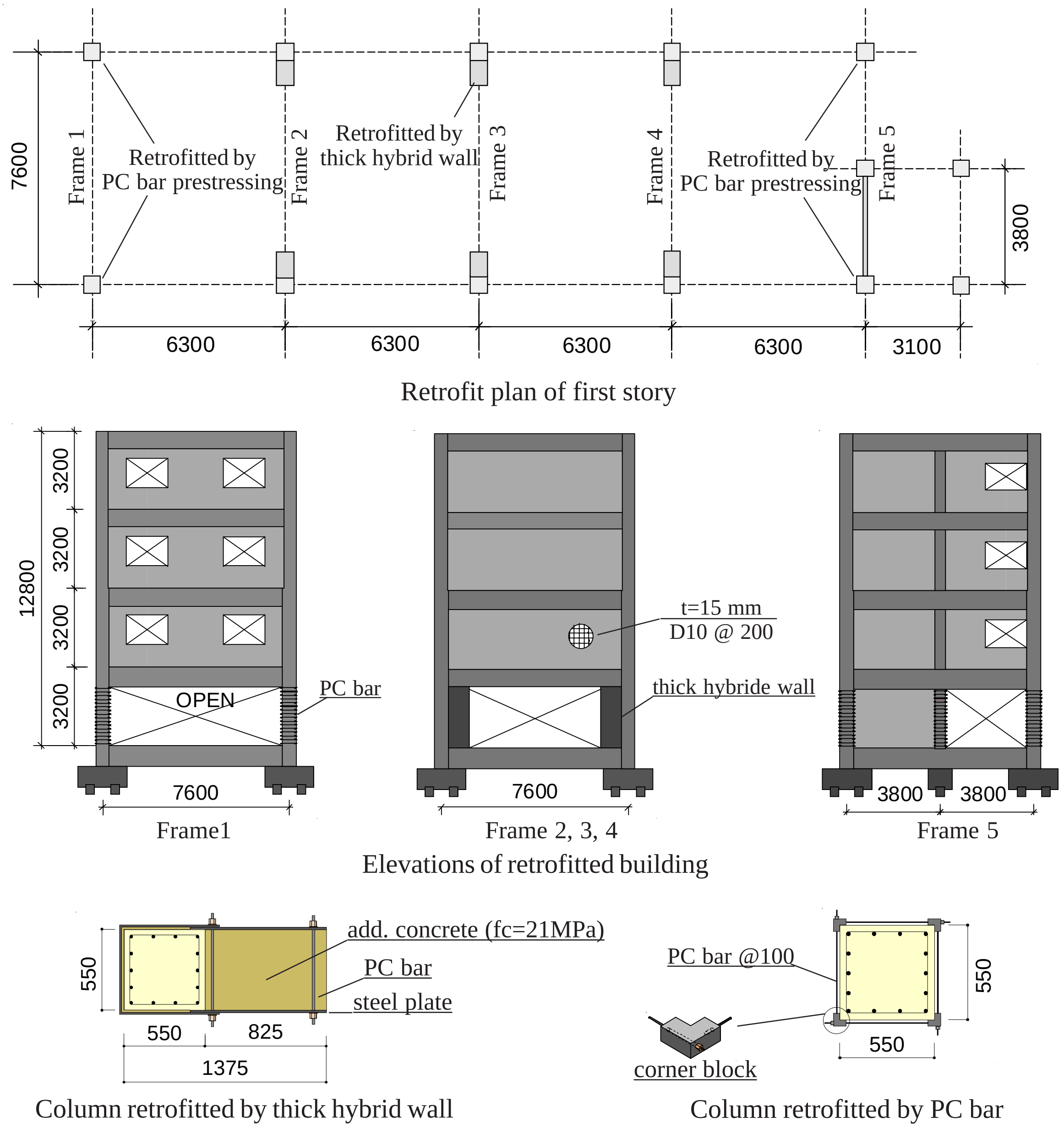



**Fig. 4.11 Details of strength-ductility-type retrofit**

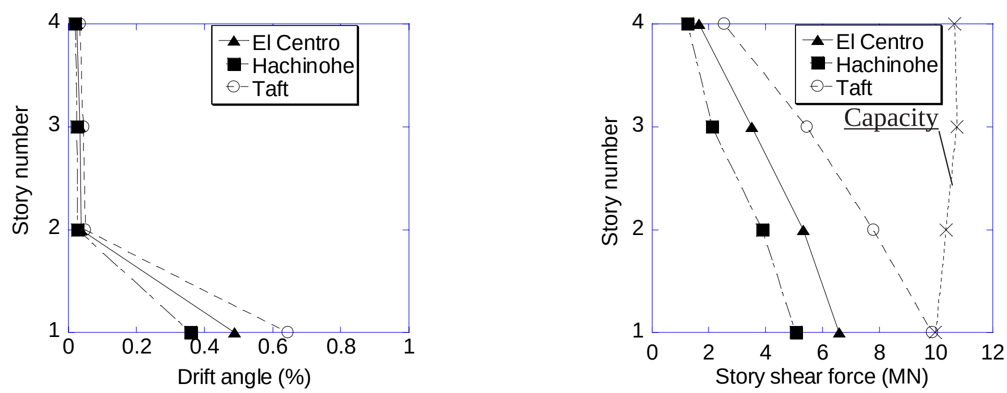



**Fig. 4.12 Drfit angle and shear force of the stories in strength-ductility-type retrofit**

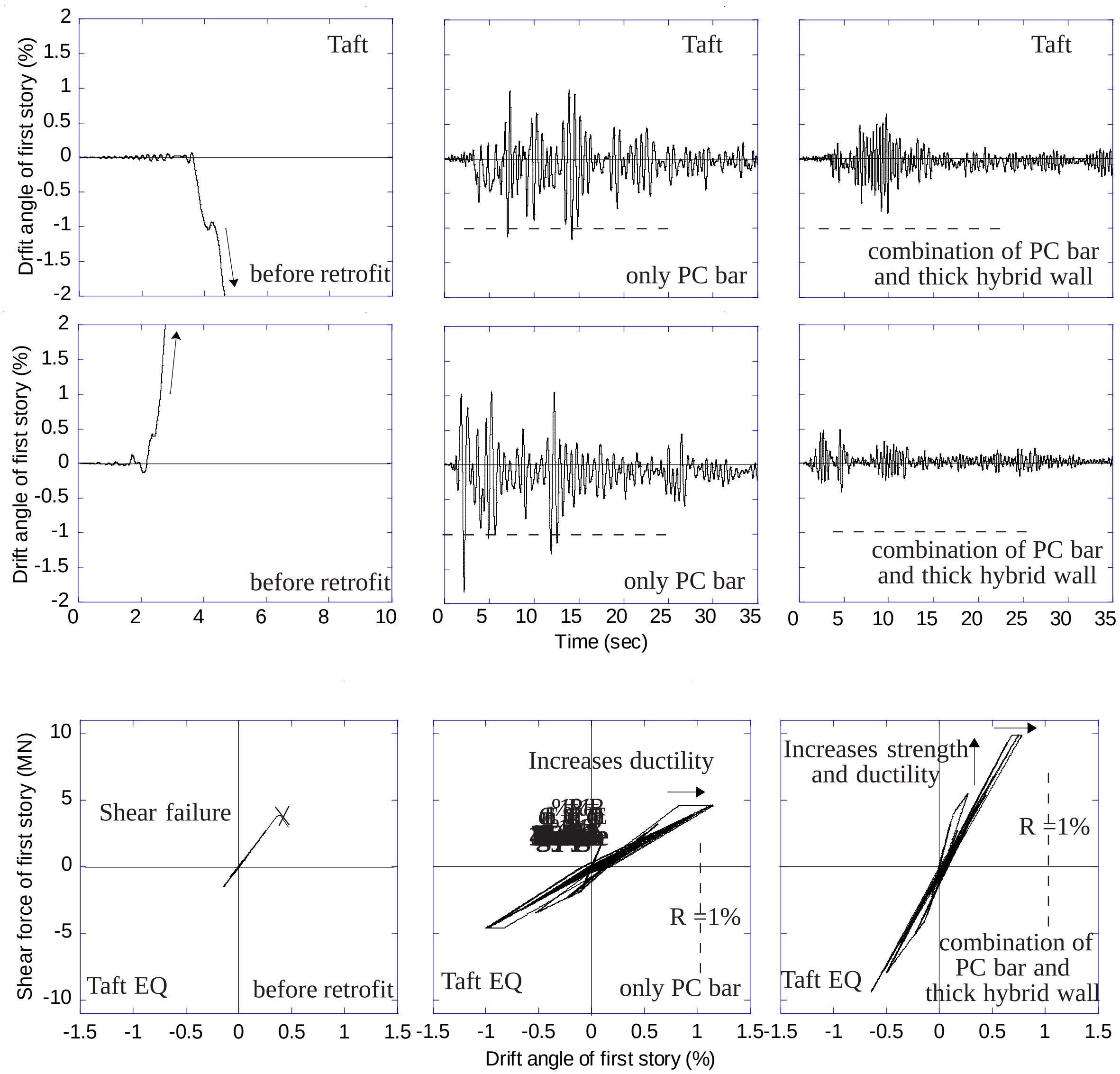



**Fig. 4.14 Hysteresis response of first story during the retrofit procedure**

## 4.9 Conclusions

The hysteresis behavior of the columns retrofitted by PC bar and thick hybrid wall can be well modeled by SINA hysteresis rule. The result of the modal analysis of the building before retrofit showed that in the pilotis building, the lateral deformation is concentrated in the first story. The dynamic analysis of the pilotis building presents that the shear failure will happen due to three basis earthquakes. Utilizing of the PC bar prestressing was found to be effective for the prevention of shear failure, but in this case the lateral deflection exceeded the maximum allowable value of 1%. Also, in retrofitting by PC bar, the comparison of results of the columns that are modeled by Takeda and SINA hysteresis rules represents that the maximum response as well as to yield strength depends on the hysteresis rules. The combination of PC bar and thick hybrid wall prevent the shear failure and increase the stiffness and strength in that the retrofitted pilotis building shows acceptable performance.

## REFERENCES


Carr, A. J. (1980-2007). "RUAUMOKO, Dynamic Nonlinear Analysis", Department of Civil Engineering, University of Canterbury, Christchurch, New Zealand.

Japan Building Disaster Prevention Association. (2001). "Standard and Guidelines for Seismic Evaluation and Retrofit of Existing RC Buildings".

Newmark NM and Rosenblueth E. (1971). "Fundamentals of Earthquake Engineering." *Prentice-Hall. Inc., Englewood Cliffs, N. J.*

Otani, S. (1981). "Hysteresis Models of Reinforced Concrete for Earthquake Response Analysis", Journal of Engineering, University of Tokyo, Vol. XXXVI, No. 2, pp. 407 - 441.

Paulay T. (1979). "Capacity Design of Earthquake Resisting Ductile Multistory Reinforced Concrete Frames." *Proc. of the 3rd Canadian Conference on Earthquake Engineering,* Vol. 2, pp. 917-948.

Rahman, M. N. and Yamakawa, T. (2004). "Experimental and Analytical Investigation of Seismic Retrofit for One-sided Wing-wall RC column", *Proceedings of the Japan Concrete Institute (JCI),* Vol.26, No.2, pp. 1351-1356.

Rahman, M. N., Yamakawa, T., Morishita, Y., and Nakada, K. (2005). "Investigation of Seismic Retrofit for Pilotis Frames Utilizing Extremely Thick Hybrid Walls", *Proceedings of the Japan Concrete Institute (JCI),* Vol.27, No.2, pp. 1117-1122.

Yamakawa, T., Li, W., Kurashige, M. (2002). "Experimental Investigation on Emergency Retrofit for Extreme Short Columns by PC Rod Prestressing", Proc. of the Japan Concrete Institute (JCI), Vol. 24, No. 2, pp. 1219-1224.

Yamakawa, T., Zhang, A. and Takaesu, M. (2001). "An Investigation on Seismic Performance of Existing Pilotis-type R/C Multiple Dwelling Houses in Okinawa." *Journal of Structural and Construction Engineering, AIJ*, No. 549, pp. 113-119. (in Japanese)

Yamakawa, T., Rahman, M. N. and Morishita, Y. (2006). "Experimental Investigation and Analytical Approach for Retrofit of RC Column with Wing-wall", J. of Structure and Construction Engg., AIJ, No. 608.


## PUBLICATION


Javadi, P., Yamakawa, T., Rahman, M. N. and Takara, S.: "Nonlinear Dynamic Response of an Existing Pilotis-type RC Building Before and After Retrofitting", *Annual Meeting of AIJ*, Vol.C-2, August, 2007. (In press)