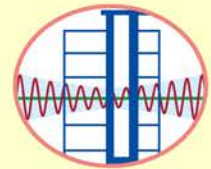


# NCREE NEWSLETTER



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**National Center for Research on Earthquake Engineering**

# Seismic Requirements for the Framing below the Base of a Building Structure

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*Gee-Jin Yu, Associate Technologist, NCREE*

The seismic design requirements for structures below a building base are mainly presented in Section 6.2.11 of the current seismic design code in Taiwan [ABRI, 2011]:

## Provision

The design strength and stiffness of the frame between the base and the foundation shall not be lower than those of the superstructure. The special provisions on the ductility of reinforced concrete structures and steel structures shall also apply to the members that transmit seismic forces from the base to the foundation. The ultimate shear force generated above the base can also be used as a lateral force, which can be applied to the base instead; however, the vertical members should still be provided with tight stirrups according to the relevant regulations on ductility.

## Commentary

The basement structure between the base and the foundation has outer walls having high stiffness, which makes it difficult for its beams and columns to yield. Therefore, although it should be able to be included in a ductile design, its strength should be sufficient and the ultimate shear strength of the ground layer should be used to designed for keeping the basement structure elastic in the event of a major earthquake. For example, the design of the ground beam should be able to withstand the ultimate shear strength of the ground floor and also an internal force caused by 1.4 times the design seismic force. When the stress method is used for design, it is permissible to not conduct a ductile design. If the shear force transmission of the first floor plan fails or the exterior wall of the basement is damaged, the ductility design of the basement members will come into effect, and the design engineer should make appropriate judgments to determine the best way to design the basement members.

The provisions and explanations of this specification refer to the content of SEAOC 1999-108.2.10 [SEAOC, 1999]. The main spirit behind the design is that the stiffness and strength of the framework below the base surface must be sufficient to withstand the full development of the superstructure's nonlinear behavior. Therefore, the design strength and stiffness of the frame below the base should not be lower than that of the superstructure, and if the design of the frame is based on the design seismic force, all its components must comply with the structural design specifications. Special provisions for seismic design stipulate that if the design is based on the ultimate shear force generated above the base level or an internal force caused by 1.4 times the design seismic force, the ductility design is not required; however, the reinforced concrete vertical members must still be provided with tight hoop stirrups.

## Summary

This manuscript clarifies the principles and procedures for ductile design or capacity design with regard to the seismic design requirements of structures below the base level of a building. Considering that the structure below the base level has no obvious ductility requirements, the strength of its vertical members should be considered in the design. Therefore, it is suggested that if the design of the frame below the base level is carried out using the design seismic force of the upper structure, not only should its strength and stiffness not be lower than those of the upper structure but its vertical members must also meet the seismic resistance requirements of each relevant structural code. It is specifically stipulated that a design should have sufficient member strength and ductility to ensure that the superstructure can effectively develop its ductility. If the ultimate shear strength above the base level and  $1.4\alpha_y$  times the design seismic force are considered, the strength design can be directly carried out. Regardless of which design procedure is adopted, however, the design of the frame below the base level still needs to consider the requirements of the aggregated members and the shear force transmission path of the floor slab, as well as the effect of non-structural walls and other factors.

In the US building design code, the superstructure and the frame below the base level are regarded as different structural systems. In other words, the frame below the base level is regarded as a structural system with low ductility capacity, and its seismic force reduction coefficient is small, making the design seismic force is greater than its superstructure, and its vertical members must be designed for capacity.

# Application of a Generalized Building Model to the System Identification of NCREE Office Building

Jui-Liang Lin, Research Fellow, NCREE  
 Gee-Jin Yu, Associate Technologist, NCREE

The post-earthquake system identification and response estimation of an elastic 13-story compound building during four seismic events were conducted using a simplified numerical model: the generalized building model (GBM). The compound building examined was the recently remodeled office building of Taiwan’s National Center for Research on Earthquake Engineering (NCREE). The 7-13th stories of the building are extended steel structures seated atop six older reinforced-concrete (RC) stories. In addition to the vertical extension, the building was horizontally extended. The extension incorporated several types of seismic dampers. The GBM, which reflects the dynamic characteristics of the elastic compound building, was constructed using only the acceleration records of the roof and the 7th story. The effectiveness of the GBM was then verified by comparing the estimated and actual acceleration responses of the other stories. From the verified GBM, the modal parameters and drift responses of the compound building were estimated accordingly.

The present study constructed the GBM reflecting the extended NCREE office building in the x direction (Fig. 1) for each of the four seismic events listed in Table 1. The shaking intensities at the 1st story of the building during events No. 2 and 4 were approximately equal. Additionally, the shaking intensities at the 1st story of the building during events No. 1 and 3 were approximately equal. The peak acceleration at the 7th story (i.e., the top of the old RC building) was approximately 1.5 to 3.5 times that at the 1st story. The peak acceleration at the roof (i.e., the top of the added steel structure) was approximately 6.5 to 8.4 times that at the 1st story. Accordingly, except for event No. 3, the amplification of the story acceleration from the top of the old RC building to the top of the added steel structure was more significant than that from the bottom to the top of the old RC building.

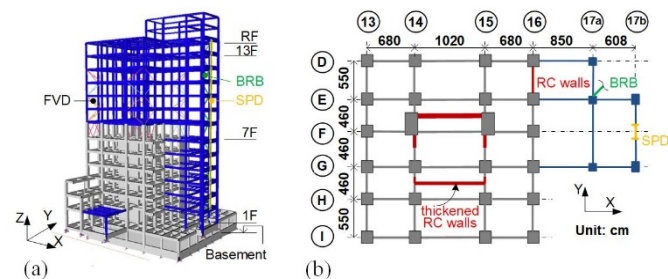


Fig. 1. (a) Perspective and (b) floor plan of the RC stories of the extended NCREE office building.

The identified modal vibration periods ( $T_1, T_2, T_3$ ) are shown in Table 2 and the damping ratios  $\xi$  were 0.029, 0.034, 0.039, and 0.032 for events No. 1 to No. 4, respectively.  $MR_i$ , where  $i = 1$  to 3, shown in Table 2 are

the modal participation mass ratios of the  $i^{th}$  mode. The mode shapes of the first three modes are shown in Fig. 2. Fig. 3 illustrates the measured and estimated acceleration histories of all stories of the building during event No. 4. Fig. 3 indicates that the GBM satisfactorily estimates the acceleration responses of the building.

Table 1. Details of the four seismic events

Event No.	Local time (UTC+8)	Magnitude ( $M_L$ )	Epicenter	Depth (km)
1	2021/04/18 22:14:37	6.2	23.86°N, 121.48°E	14.4
2	2021/10/24 13:11:34	6.5	24.53°N, 121.78°E	65.6
3	2021/12/30 14:47:09	5.5	23.98°N, 122.51°E	29.7
4	2022/01/03 17:46:37	6.0	24.02°N, 122.18°E	19.4

Table 2. Identified modal parameters of the building for the four seismic events

Event No.	1 <sup>st</sup> mode		2 <sup>nd</sup> mode		3 <sup>rd</sup> mode	
	$T_1$ (s)	$MR_1$	$T_2$ (s)	$MR_2$	$T_3$ (s)	$MR_3$
1	0.760	0.575	0.266	0.169	0.182	0.157
2	0.810	0.592	0.287	0.186	0.197	0.136
3	0.760	0.624	0.275	0.203	0.188	0.098
4	0.830	0.575	0.290	0.169	0.198	0.157

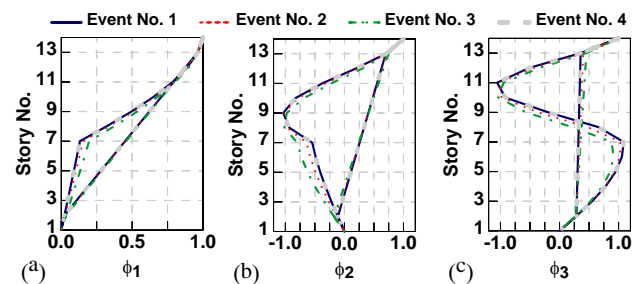


Fig. 2. The first three mode shapes (a)  $\phi_1$ , (b)  $\phi_2$ , and (c)  $\phi_3$  of the building for the four seismic events.

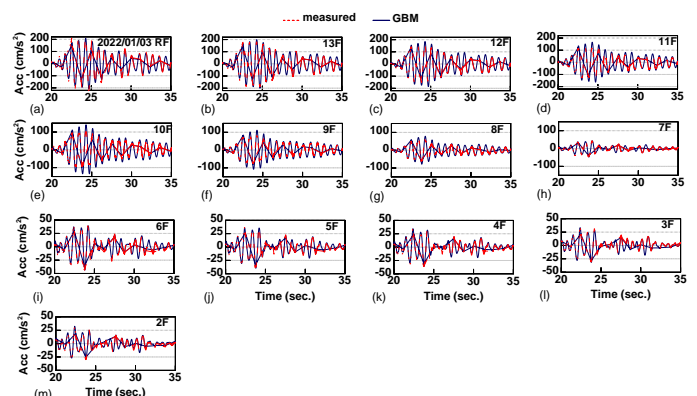


Fig. 3. The measured and estimated acceleration histories of all stories of the building during event No. 4.

# Seismic Behaviors of High-Strength KC Columns under High Axial Loads

Ker-Chun Lin, Research Fellow, NCREE  
 Kai-Ning Chi and Sheng-Jhih Jhuang, Assistant Researcher, NCREE

Traditionally, a reinforced-concrete (RC) column places its longitudinal reinforcements on the periphery of the column section to maximize flexural strength capacity. For RC moderate- and high-rise buildings, columns at lower stories usually accumulate the huge self-weight of the building, so a larger number of longitudinal reinforcements need to be provided to resist the required axial load. This may, however, result in excessively close placement of longitudinal reinforcements and may reduce the workability of pouring concrete and increase the difficulty of placing reinforcements of the intersected beam. Under high axial load and seismic loading reversals, the longitudinal reinforcements on the periphery of the column section easily buckle after the column cover peels out, further degrading the column strength. To reduce the effect of buckling of the longitudinal reinforcements on the periphery of the column section, some part of the longitudinal reinforcements may move to the core of the column, called the ‘kernel-confined (KC) bar’, which is expected to prevent the reinforcement from buckling or to decrease the effect of strength degradation due to buckling. Use of a KC bar in a column (KC column) can reduce the number of longitudinal bars on the periphery of columns, facilitate the quality of concrete pouring, and delay strength degradation of the column due to rebar buckling.

## Design of Transverse Rebar Confinement

For the existing Concrete Structures Design Code in Taiwan (CPAMI 2019) and ACI 318-11 (ACI 2011), the confined transverse reinforcement ratio  $R_t (= A_{sh}/s_b c)$  shall satisfy the larger of:

$$R_{t,(a)} = 0.3 \frac{f'_c}{f_{yt}} \left( \frac{A_g}{A_{ch}} - 1 \right) \quad (1) \quad \text{or} \quad R_{t,(b)} = 0.09 \frac{f'_c}{f_{yt}} \quad (2)$$

When a column is subjected to axial load exceeding  $0.3f'_c A_g$ , achieving a lateral drift ratio of 3% rad may not be ensured. According to the ACI 318-14 and -19 codes, in addition to satisfying Eqs. (1) or (2), a requirement for a confined transverse reinforcement ratio for columns by considering the effects of high-strength concrete and high axial load:

$$R_{t,(a)} = 0.2k_f k_n \frac{P_u}{f_{yt} A_{ch}}, \quad k_f = \frac{f'_c}{175} + 0.6 \geq 1.0, \quad k_n = \frac{n_l}{n_l - 2} \quad (3)$$

was included to acquire the expected deformation capacity of an interstory drift angle of 3% rad for columns.

## Experimental Plan

In this research, six column specimens with high-strength RC materials were subjected to a high axial load of  $0.5f_{ca}' A_g$  to investigate the lateral seismic performance of KC columns. The design compressive strength of concrete was 70 MPa and the longitudinal and transverse reinforcements used SD 550W and SD 790 (USD 785

from Tokyo Tekko Company, Japan), respectively. The cross-section dimensions and height of the specimen were 600 mm × 600 mm and 1800 mm, respectively, and the height-to-width ratio was 3. The design parameters and results and placement of longitudinal and transverse reinforcements for all specimens are shown in Figure 1.

Specimen type	NCK05BP5	NCK38BP5	NCK28BP5	NCK45BP5	TCK05BP5	TCK08BP5
$f'_c$ (MPa)	70					
Grade	SD 550					
Longitudinal	Periphery Reinforcements	20 #8	20 #8	16 #8	12 #8	12 #8
	Kernel Reinforcements	0	4 #11	4 #8	4 #11	4 #11
	$\rho$	0%	28%	20%	40%	40%
Transverse	Longitudinal ratio $\rho$	2.62%	3.93%	2.62%	2.62%	2.62%
	size and spacing	$\phi 4 @ 80$	$\phi 4 @ 80$	$\phi 4 @ 80$	$\phi 4 @ 80$	$\phi 4 @ 80$
	$f'_c$ (MPa)	690	690	690	690	790
$\rho$	1.01%	1.01%	1.01%	1.01%	0.88%	
$\rho$	0.91%	0.91%	0.91%	0.91%	0.80%	
$\rho$	1.62%	1.62%	1.69%	1.30%	1.57%	

Fig. 1. Design parameters of specimens.

## Test Results and Discussions

1. Experimental results (Figure 2) showed that the maximum shear responses of the column specimens were achieved at their shear capacities. The hysteresis curves of the relationship between shear and deformation for all specimens demonstrated relatively plump hysteresis behaviors and no obvious pinching.
2. The flexural strengths predicted by XTRACT were accurate with an average error of 1%. The flexural strengths predicted by the ACI model were underestimated with an average of 25%.
3. Under a high axial load of  $0.5f_{ca}' A_g$ , the deformation and strength capacities for the KC column specimens with 20% to 40% KC bars approximated those for the conventional column. It was found that providing a confinement of  $R_{t,(c)}$  was a feasible criterion to ensure that a column member has a lateral deformation angle of 3% rad. Therefore, if a column member has  $R_{t,(c)}$  confinement, then some of the closer longitudinal bars for the conventional column on its periphery are able to move to the kernel region as KC bars, thereby easing the proximity of longitudinal reinforcements, which reduces the number of tie bars and enhances the quality of pouring concrete.

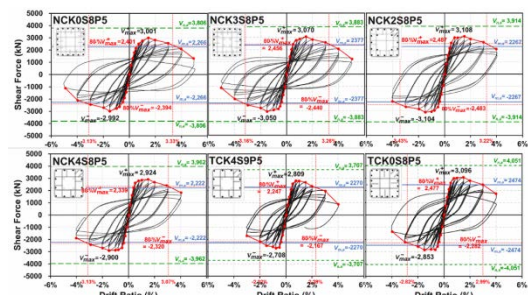


Fig. 2. Hysteretic loop curves between column shear and interstory drift angle for all specimens.

## Seismic Evaluation of Steel Buildings (TEASPA-S)

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In Taiwan, detailed evaluations of the seismic capacities of reinforced concrete building structures are very common, both in the study of assessment methods and in engineering practice. However, with the gradual aging of buildings and the continuous updating of design specifications, the seismic capacities of existing steel structures in Taiwan may be insufficient. In recent years, the number of new buildings using steel structures has increased, and there will be greater need for earthquake resistance assessment and reinforcement of steel structures in the future.

The detailed evaluation method of seismic capacity of a steel structure developed in this research is a continuation of the principles of the National Center for Research on Earthquake Engineering (NCREE) detailed evaluation method of the seismic capacity of a reinforced concrete building (TEASPA, the Taiwanese earthquake assessment for structures by pushover analysis). The evaluation method is based on the Applied Technology Council ATC 40 capacity spectrum method and nonlinear static pushover analysis, and uses the ETABS program commonly used by domestic engineers to perform nonlinear static pushover analysis to obtain the capacity curve of a structure. Then, it is converted into a seismic performance curve through the capacity spectrum method in order to evaluate the seismic performance of the structure.

This study has completed the evaluation of the moment-resisting frame and the concentric braced frame. Of the structural members used in the evaluation, the steel columns and steel beams use the nonlinear hinge properties recommended by the American Society of Civil Engineers ASCE 41-13 standard, and the concentric braces use the nonlinear hinge properties developed in this study.

In the Northridge earthquake in the United States in 1994, many of the damaged steel structures exhibited mainly brittle failure of beam-column joints, and the plastic deformation angle of some beam-column joints was less than 0.015 rad. In 2000, the Federal Emergency Management Agency (FEMA 350) proposed a new form of certified joint (Prequalified Connections) to improve the ductility performance of beam-column joints. For this reason, the evaluation method developed by this research considers two joint types, traditional joints and improved joints, and develops recommended seismic performance criteria for each.

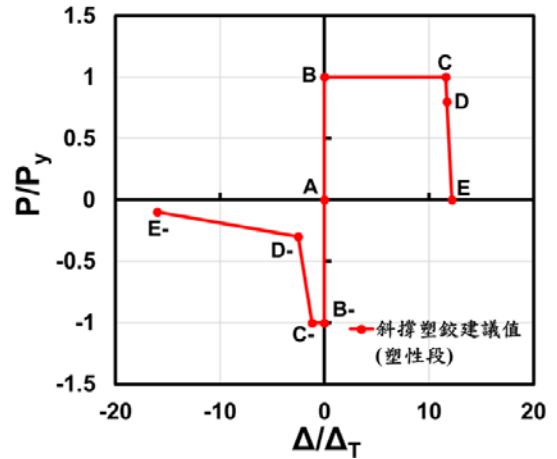


Fig. 1. Axial force versus displacement of a concentric brace.

A graph for the concentric diagonal bracing plastic hinge suggested in this study is shown in Fig. 1, and the suggested values for the plastic hinge are given in Table 1. The suggested values for the plastic hinge were obtained by collecting test data of existing diagonal bracing members in Taiwan and conducting statistical analysis, and considering the convergence problem of the ETABS program.

Table 1 Nonlinear parameters of a concentric brace.

Point	Force/SF	Disp/SF
E-	-0.1	-16
D-	-0.3	-2.5
C-	-1	-1.2
B-	-1	0
A	0	0
B	1	0
C	1	11.6
D	0.8	11.72
E	0	12.2

This study has already proposed eleven inspection items and methods that should be carried out in seismic assessment. These will assist engineers in ensuring the reasonableness in avoiding misinterpretation of seismic assessment results, in order to maintain their accuracy.

# Analysis of the Connections Joining the New Steel Structure and the Existing RC Structure in the Extended NCREE Building

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Yu-Cheng Lin, Former Research Assistant, NTU

Keh-Chyuan Tsai, Professor, NTU

Jui-Liang Lin, Division Director of the Building Engineering Division, NCREE

The project involving vertical additions to the National Center for Research on Earthquake Engineering (NCREE) building began in March of 2019. The extended NCREE building (Fig. 1) is a composite structure that includes an existing six-story reinforced concrete (RC) structure and an added seven-story steel structure. A new service core was also added at the north side of the building from the first floor to the roof. To sustain the added steel structure, the existing RC structure was strengthened using RC shear walls and fiber-reinforced polymer strengthened RC beams. Buckling-restrained braces, steel panel dampers, and fluid viscous dampers were utilized in the added steel structure to improve the seismic performance.

To understand the seismic performance of the extended NCREE building, a collaborative research project was conducted by NCREE researchers and Prof. Keh-Chyuan Tsai's research group from the National Taiwan University. PISA3D structural analysis software was used for constructing the numerical model (Fig. 1) to conduct nonlinear response history analyses (NRHAs) (Fig. 2). These were conducted for the NCREE's thirteen-story composite building in order to gain insight into the effects of possible maximum seismic forces and deformational demands. A total of sixteen sets of ground accelerations were utilized. The NRHA results indicate that the maximum axial force and maximum in-plane rotation of the connection joining the new steel structure and the existing RC structure are approximately 165 tf and 0.01 radians, respectively.

Abaqus finite element analysis (FEA) software was used to examine the connection types. Based on the FEA results (Fig. 3), the strengthened connection, which consisted of double-sided stiffened steel angles at the beam web and a straight bracket with angular cuts on the cover plates, was deemed to be suitable. Thus, the strengthening scheme was widely used in the extended NCREE building (Fig. 4).

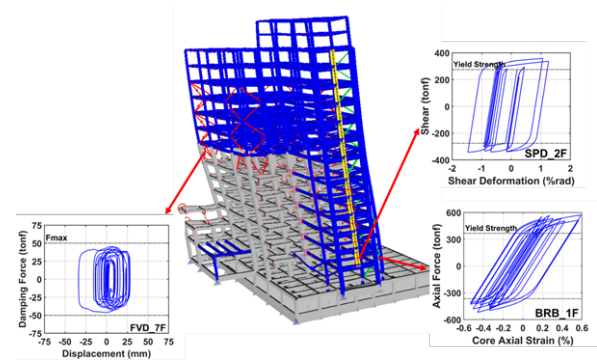
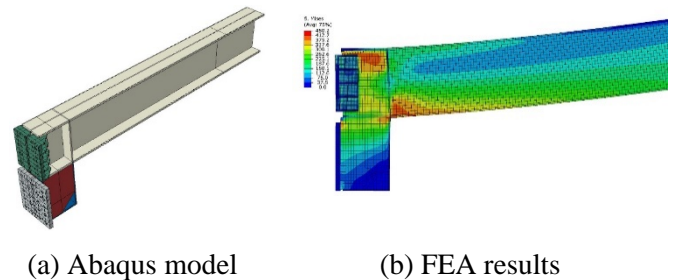


Fig. 2. Perspective view of the deformation shape (as the numerical model reached the peak roof displacement) and hysteresis loops for the dampers



(a) Abaqus model

(b) FEA results

Fig. 3. The connection consisted of double-sided stiffened steel angles at the beam web and a straight bracket with angular cuts on the cover plates

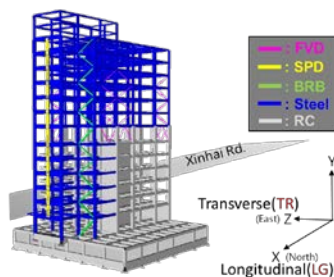


Fig. 1. PISA3D model of the extended NCREE building



Fig. 4. The connection of the strengthening scheme

# Truss-confined Buckling-Restrained Braces

An-Chien Wu, Associate Researcher, NCREE

## Assembly of TC-BRB

Recently, a novel type of buckling-restrained brace (BRB) called truss-confined BRB (TC-BRB) with a varying cross-sectional depth has been investigated. The restraining system of the proposed TC-BRB is fabricated using several truss frames attached to a common steel casing (Fig. 1). Each truss frame is symmetric relative to its mid-span and composed of several truss segments in series. Each of the truss segments consists of a chord

almost parallel to the BRB longitudinal axis, a diagonal, and a pair of posts approximately in the BRB's transverse direction. Most of the flexural rigidity of the truss system is provided by the chord, while the diagonal and posts mainly contribute the overall shear rigidity of each segment. In order to improve the out-of-plane stability of the truss-confining system, transverse components are distributed at a certain distance along the BRB longitudinal axis to connect the truss frames. The flexural and shear rigidities contributed by the transverse components are negligible.

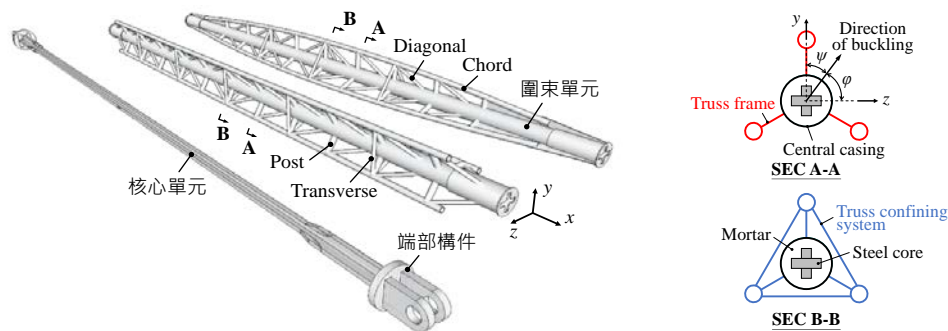


Fig. 1. Schematic of the TC-BRB.

## Seismic Performance of TC-BRB

In order to represent the TC-BRB as a part of the mega braced frames adopted in a tall building, an approximately 45-degree-inclined diagonal brace across four stories, each with a story height of 4m, was considered. Due to the limitation of the testing facility, the prototype TC-BRB in the mega braced frame was scaled by approximately 1:3 (Fig. 2). As a result, the TC-BRB specimens were designed with a total length of approximately 6.3m with a nominal yield capacity of 850kN. The specimens were tested using the multi-axial testing system (MATS) at the National Center for Research on Earthquake Engineering

(NCREE) by applying cyclically increasing displacements as prescribed in AISC 341-16. The standard loading proceeded with increasing axial deformations considering inter-story drift ratios (IDRs) ranging from 0.5% to 2% for two cycles at each deformation level. The extended loading was continued with five cycles of 2% IDR to achieve an index of 200 cumulative plastic deformation (CPD) as prescribed in the specifications. In order to trigger possible instability of the specimens, axial deformations corresponding to 3% to 5% IDRs were applied following the last cycle of 2% IDR until failure occurred.

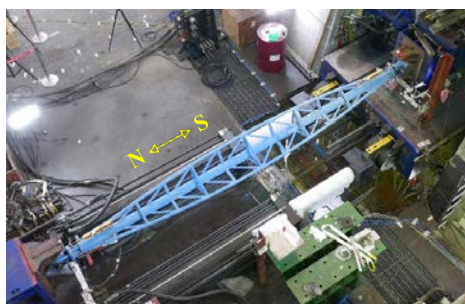


Fig. 2. Experimental setup.

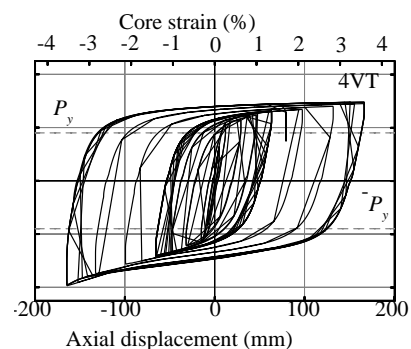


Fig. 3. Hysteretic response of the specimen.

The experimental performance demonstrated satisfactory hysteretic behavior complying with the requirements prescribed in AISC 341-16 (Fig. 3). Test results indicate that a properly fabricated TC-BRB specimen designed with a stiffer truss-confining system is able to sustain a larger CPD without flexural buckling. It is confirmed that the proposed TC-BRB can be designed

and fabricated to achieve excellent seismic performance. A simplified method of estimating the elastic buckling strength of the TC-BRB was developed. Its accuracy was verified through finite-element model analysis. Recommendations on the seismic design and analysis of the TC-BRB will be introduced in a paper that is currently being prepared.

# Taiwan Earthquake Assessment for RC Structures by Dynamic Analysis (TEASDA 1.0)

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 Wen-Cheng Shen, Assistant Researcher, NCREE  
 Fu-Pei Hsiao, Research Fellow, NCREE

The Meinong earthquake struck southern Taiwan in the early morning of February 6, 2016. Two years later, a major earthquake occurred in the Hualien area at midnight on February 6, 2018. Several mid-to high-rise buildings collapsed after each earthquake. To effectively address this problem, this article, in conjunction with the earthquake duration screening database recently recommended by the National Center for Research on Earthquake Engineering (NCREE), proposes a set of easy-to-implement non-linear dynamic analysis methods (called "Non-linear dynamic analysis methods for seismic evaluation of reinforced concrete structures in Taiwan" (Taiwan Earthquake Assessment for Structures by Dynamic Analysis, TEASDA) (Figure 1)), as the standard reference for domestic and foreign industry, academia, and research circles. It is hoped that this can more effectively clarify doubts about the earthquake resistance of existing buildings and serve as a basis for subsequent improvement of earthquake resistance.

has three failure modes, shear failure, flexure-shear failure, and flexural failure. Li *et al.* (2019) found that the deformation capacity of a shear dominant column was not as conservative as expected and suggested a model presenting the lateral load-displacement curve. When a column has sufficient stirrups, the column shear strength will be higher than the flexural strength, and its failure mode will tend to flexural behavior. Since the position of the inflection point of the column will change during the nonlinear reaction of the frame, the failure mode of the column cannot be specified before the analysis. Therefore, in the setting of nonlinear hinges of a column member, it is necessary to set the moment nonlinear hinges at both ends of the column, as shown in Fig. 2.

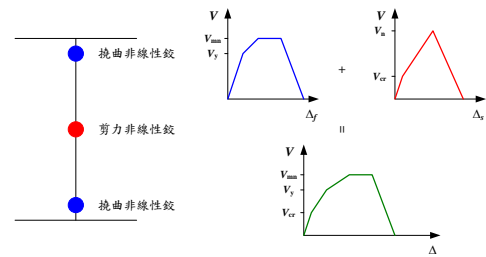


Fig. 2. Behavior and simulation of a column

## Hysteresis Model of Structural Members

Nonlinear dynamic analysis involves repeated loading. The hysteresis behavior of a nonlinear plastic hinge of a structural member must be clearly defined, including the characteristics of unloading and reloading. An appropriate hysteresis model must be selected to describe the energy dissipation behavior of different structural members. This study proposes the use of the Pivot hysteresis model for an RC column, in which the force-displacement curve points to specific reference points during unloading and reloading. These reference points are called "pivot points", and defining these pivot points with parameters  $\alpha$  and  $\beta$  can describe the hysteresis behavior of nonlinear plastic hinges. Yu-Che Ling (2019) suggested that the two parameters can be calculated according to the member characteristics.

## Conclusions

Mid-to high-rise buildings are characteristically associated with a high population density. Once a disaster occurs, the risk to human life and potential damage to property is much greater than that of low-rise buildings. In order to effectively address this problem, this paper proposed an easy-to-implement nonlinear dynamic analysis method as a reference for domestic and foreign industry, academia, and researchers, in the hopes of more effectively clarifying the seismic capacity of existing buildings, and removing any ambiguities in the subsequent improvement of earthquake resistance.

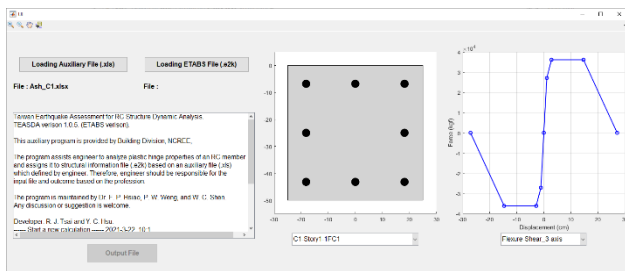


Fig. 1. Taiwan Earthquake Assessment for Structures by Dynamic Analysis Software (TEASDA)

## Ground Motion Selection

The NCREE built a database of "Recorded Ground Motion Selection for Generic Sites" (Liu Xunren *et al.*, 2020) to enable engineers to conduct design checks and seismic assessment following the relevant requirements of the seismic code when conducting dynamic analysis. At present, the platform "Input Motion Selection for Taiwan" has been preliminarily constructed for earthquake-prone areas in Taiwan in general and in the Taipei Basin (excluding near-fault areas). This platform can assist in the selection of earthquake records for design verification.

## Member Behavior and Simulation

Engineers often use structural analysis packaged software such as ETABS and SAP2000 to perform pushover analysis or nonlinear dynamic analysis of reinforced concrete (RC) structures. Therefore, it is necessary to understand the nonlinear behavior of structural members, such as columns, beams, and RC walls, before assigning structural members suitable nonlinear hinge properties and positions to simulate their nonlinear behavior. For example, an RC column is subjected to both an axial load and a lateral load, and it

## Revision of the Design and Technique Specifications for Steel Structure Buildings

*Sheng-Jih Jhuang and Kai-Ning Chi, Assistant Researcher, NCREE  
Ker-Chun Lin, Research Fellow, NCREE*

Because of their high strength, light weight, good toughness, superior performance, lively shape, fast construction, and use of a recyclable and reusable green building material, steel structure buildings have become the main choice for general buildings. Their sectional structure and method of connection need to be properly analyzed and designed, and their constructability and material specifications must also be considered. In addition, attention still needs to be paid to their durability and fire resistance. Therefore, the design operation of steel structure buildings needs to be completed through detailed planning and analysis, and rigorous and complicated calculation procedures. Hence, the formulation of relevant standards and specifications is very important.

The domestic steel structure building design specification began in 1999. According to the legal source of "Building Technical Regulations," the name of the specification was "Design and Technique Specifications of Steel Structures for Buildings," which was divided into "Specification and Commentary of Allowable Stress Design Method for Steel Structures" and "Specification and Commentary of Ultimate Strength Design Method for Steel Structures." This specification was revised in 2007 and has been used for more than 14 years. During this period, domestic steel structure engineering technology has also changed and improved to a considerable extent, including changes in material regulations, manufacturing, hoisting and erecting, and inspection methods. Good and suitable steel structure design specifications are also related to the safety and development of high-rise buildings in Taiwan. In this regard, it was conceived to develop a new draft of "Design and Technique Specifications of Steel Structures for Buildings" that integrates the revision direction and content of recent domestic and foreign steel structure technology and specifications, as well as the development and research results of steel structure construction technology.

The establishment of the domestic steel structure design specification is mainly based on the design specification published by the American Institute of Steel Construction (AISC). The allowable stress design method and the ultimate strength design method of the current steel structure design specifications have been revised according to the Allowable Stress Design (ASD) method of AISC in 1989 and the Load and Resistance Factor Design (LRFD) method of AISC in 1999. Compared with the current 2016 version of AISC 360 (Specification for Structural Steel Buildings) and AISC 341 (Seismic Provisions for Structural Steel Buildings), these are already 17 years old. Moreover, the new revised versions of AISC 360 and 341 are coming to an end, and they are expected to be re-published in 2022. By then, the age difference will be more than 20 years. During this period, more advanced steel structure design concepts, methods, and systems have been developed and incorporated into the AISC 360 and 341 specifications. Therefore, this research will discuss and study the revision of the domestic steel structure design specification in order to incorporate advanced steel structure design technology.

To maintain consistency in the current specifications, this revision of the domestic steel structure design specification is based on the 2016 version of the AISC 360 and 341, and revised with reference to domestic engineering practices and research results. In this study, the two volumes of the allowable stress design method and the limit design method of the current specification were combined into one volume. A new allowable strength design method will replace the allowable stress design method of the current specification, and based on the LRFD design method combining the ultimate strength of a member with the probability-based load criterion as the revised standard, the allowable strength design method and the LRFD design method have the same safety level design results. To improve the effectiveness of the seismic design of structural steel buildings, this revised edition introduces the expected actual strength concept of materials in the Seismic Design section of Chapter 14 as the design basis for the strength capacity of the members or elements. It also aims to ensure that the actual seismic behavior of a building structure will be as consistent as possible with the design assumptions, and to drive the ductility of the seismic energy dissipation components to develop as expected. In addition, this revised draft also adds domestic engineering practical requirements and seismic systems with high-efficiency seismic performance, including seismic structural systems such as cantilever columns, buckling-restrained braces (BRB), and special plate shear walls, and other related seismic design regulations. In the research and development of this revised edition, chapters on the verification test regulations for seismic structures, components, connections, and frames were also added in an effort to make the verification test methods and procedures consistent with the qualification standards. This can also be used to verify the effectiveness of seismic structures, components, joints, or frames designed or developed by designers or developers. The requirements for the connection of steel hollow structural section (HSS) pipes have also been significantly expanded in this revised edition. This part is helpful for the development of localized design and construction technology for offshore wind power supporting structures in Taiwan.

The draft revision work of this steel structure design specification has been carried out by a collection of domestic technicians, scholars, and researchers who have knowledge and expertise that have voluntarily participated to form the "Revision Draft of Steel Structure Design Specification" committee for execution. The research on this draft was conducted through a chapter-by-chapter and article-by-article discussion procedure. Each committee member devoted their free time to study the content of each chapter, and more than 40 article-by-article discussion meetings were held to achieve the goals of the committee such as a rigorous formulation of the provisions, completeness of specifications, domestic applicability, and consistency. With the selfless dedication and hard work of the participating members of the committee, this revised draft was completed and we have taken a practical step toward the revision of the national steel structure design specification.

# A Force-Displacement Model for Shear-Critical Reinforced Concrete Columns

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 Shyh-Jiann Hwang, Professor, NTU  
 Yi-An Li, Assistant Professor, NCHU  
 Pu-Wen Weng, Associate Researcher, NCREE

Many densely populated urban regions around the world contain buildings in which the first floor is designed for commercial use or parking and the upper floors are designed for residential use. Due to the requirements of open space, there may be almost no walls on the ground floor of these buildings. If the columns are reinforced concrete and they are not detailed for earthquake resistance, they may be vulnerable to shear damage during strong earthquake ground shaking (Fig. 1), which may in turn lead to collapse of the building.



Fig. 1 Failure of a shear-critical column

The ASCE/SEI 41-13 does not incorporate the shear deformation caused by shear crack expansion in the lateral displacement of the strength point. Hence, the lateral displacement appears to be underestimated. The shear crack expansion within the shear failure regions should be taken into consideration when modeling the lateral force-displacement relationship. Two types of shear failure modes within the shear failure regions have been identified (Fig. 2). Evaluation of the shear strength for shear compression failure is based on the softened strut-and-tie (SST) model:

$$V_{n,c} = C_d \cos \theta = K \zeta f'_c A_{sr} \cos \theta \quad (1)$$

The shear strength for shear tension failure can be calculated using the shear strength empirical equation of ASCE/SEI 41-13 given below:

$$V_{n,t} = k \left[ \frac{A_v f_{yt} d}{s} + \left( \frac{0.5 \sqrt{f'_c}}{M/Vd} \sqrt{1 + \frac{N}{0.5 \sqrt{f'_c} A_g}} \right) 0.8 A_g \right] \quad (2)$$

The shear strength of column ( $V_{n,s}$ ) is the minimum value of the above shear strengths. The seismic performance curve of shear failure reinforced concrete columns can be described by the trilinear lateral force-displacement relationship including the cracking point, the strength point, and the collapse point, as shown in Fig. 3.

Figure 4 compares experimentally determined lateral force-displacement envelopes with calculated envelopes for columns that failed in shear. Before the shear strength of column is reached, there is a significant reduction in stiffness. Evaluation of the test data shows that this stiffness reduction may be attributed to the expansion of the shearing crack. The calculated lateral force-

displacement envelopes provide a good estimate of the test envelopes.

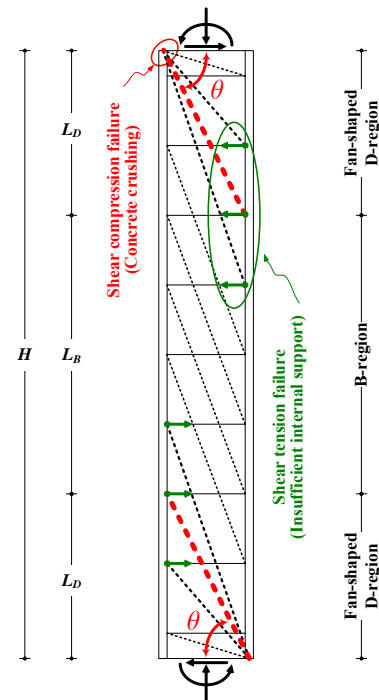


Fig. 2 Force transfer mechanism of column

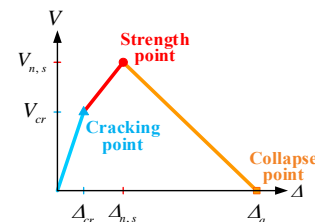


Fig. 3 Lateral force-displacement curve of column failed in shear

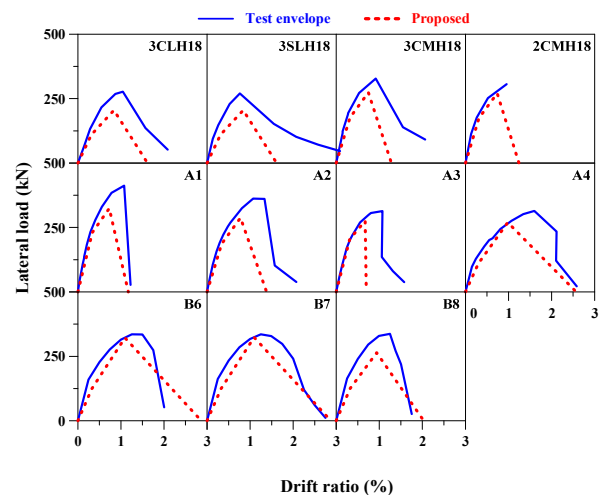


Fig. 4 Comparison of lateral force-displacement curves

# Comparison of Specifications of SRC Column and Beam-to-Column Connections in Taiwan with those in America and Japan

*Yu-Fang Liu, Associate Researcher, NCREE*  
*Te-Kuang Chow, Associate Technologist, NCREE*  
*Chung-Che Chou, Director General, NCREE*

Based on both domestic and foreign specifications and research results, this study explores the design of steel-reinforced-concrete (SRC) column and beam-to-column connections to facilitate revisions of design provisions as a reference for the Architecture and Building Research Institute, Ministry of the Interior, ROC (Taiwan). Compared with those specified in AISC 360-16 (2016) and AISC 341-16 (2016), the width-to-thickness ratio in Specifications of SRC in Taiwan (2011) is less conservative. Furthermore, three types of beam-to-column connections are mentioned in AISC 341-16 (2016). Among them, the connections of RC columns and steel beams have been included in both American and Japanese specifications but not in Taiwanese specifications. This study focuses on design theories of RCS structure, and compiles the proposed shear strength of connection's calculation method for reference.

Comparison of the Specifications of SRC in Taiwan (2011) and AISC 341-16 (2016) indicates that the specified compressive strength of concrete ( $f_c'$ ) and the minimum yield stress of the steel bars should be increased from 210 to 280 kgf/cm<sup>2</sup> and 3520 to 4200 kgf/cm<sup>2</sup>, respectively. Moreover, the length of longitudinal reinforcement in the SRC column is stipulated to be less than 300 mm. According to the Design Specifications for Concrete Structures (2021), it is suggested that the length of longitudinal reinforcement in the column should include the minimum value limit, and the length is the maximum value among 4 cm, 3/2 times the diameter of longitudinal reinforcement in the column, and 4/3 times the maximum particle size of the coarse aggregate. Table 1 compares the width-to-thickness ratios of filled composite sections according to the Specifications of SRC in Taiwan (2011) and AISC 341-16 (2016). Chou *et al.* (2019, 2020) suggested revising  $\lambda_{pd}$  to the value specified in AISC 341-16 (2016). In the Specifications of SRC in Taiwan (2011), the width-to-thickness ratio constitutes  $\lambda_{pd}$  and  $\lambda_p$ . It is suggested to retain  $\lambda_p$  and replace  $\lambda_{pd}$  with  $\lambda_{md}$  and  $\lambda_{hd}$ . The values of  $\lambda_{md}$  and  $\lambda_{hd}$  shown in Table 2 are

converted by ratio according to the values for the rectangular concrete filled tube shown in Table 1.

Three types of beam-to-column connections are mentioned in AISC 341-16 (2016). The first type is the welded joint between the encased composite column and the steel column; it is suggested that the strength can be conservatively calculated on the basis of the strength of the steel column and steel beam connection. The second type is the beam-to-column connection between filled composite columns. The third type is the connection between the RC column and the steel beam. In AISC 341-16 (2016), the method of calculating the shear strength of this type of connection adopts the superposition principle. The shear strength reduction factor of the steel frame is 0.9 and that of the RC column is 0.75.

In the AIJ Standards for Structural Calculation of Steel Reinforced Concrete Structures (2014), the design strength of an SRC structures' connections is based on allowable stress and the superposition principle. Here, the calculation formula for the shear strength of an SRC connection is divided into long-term load and short-term load. Considering the cracks in concrete caused by a long-term load, the shear strength of an SRC connection is controlled by the crack resistance of concrete. Meanwhile, the shear strength of concrete, reinforcement, and steel web under a short-term load should be taken into consideration to calculate the design shear strength of beam-to-column connections to prevent overall shear failure.

In conclusion, this study briefly introduced the design recommendations of column and beam-to-column connections in AISC 341-16 (2016) and AIJ Standards for Structural Calculation of Steel Reinforced Concrete Structures (2014). Accordingly, it is suggested that the Specifications of SRC in Taiwan should include relevant regulations by referring to AISC 341-16 (2016) and making the specifications more comprehensive.

Table 1. Comparison of Width-to-Thickness Ratios of Composite Filled Sections



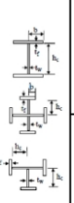
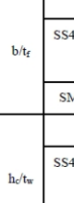
Limitation of Filled Composite Sections' Width-to-Thickness Ratio						
Width-to-Thickness Ratio	Type of Steel	Suggested Revision			Specification (2011)	
		$\lambda_{hd}$	$\lambda_{md}$	$\lambda_p$	$\lambda_{pd}$	$\lambda_p$
	SM570	30	49	55	38	-
	SS490 · SM490 · SN490	35	56	61	43	61
	SM400 · SN400	38	61	72	50	72
	SM570	36	72	86	62	-
	SS490 · SM490 · SN490	48	96	109	70	109
	SM400 · SN400	56	111	150	82	150

Table 2. Comparison of Width-to-Thickness Ratios of SRC Columns

Limitation of SRC Columns' Width-to-Thickness Ratio						
Width-to-Thickness Ratio	Type of Steel	Suggested Revision			Specification (2011)	
		$\lambda_{hd}$	$\lambda_{md}$	$\lambda_p$	$\lambda_{pd}$	$\lambda_p$
	SM570	9	14	16	10	-
	SS490 · SM490 · SN490	11	18	20	12	20
	SM400 · SN400	12	19	23	14	23
	SM570	35	57	64	60	-
	SS490 · SM490 · SN490	46	74	81	68	81
	SM400 · SN400	50	80	96	79	96
	SM570	35	57	64	60	-

# Study on Development Length in Tension and Bond Splitting Performance of Concrete for Rebar

Kai-Ning Chi, Assistant Researcher, NCREE  
 Ker-Chun Lin, Research Fellow, NCREE  
 Sheng-Jih Jhuang, Assistant Researcher, NCREE

## Introduction

In the current domestic design specifications for concrete structures, the formula for the development length of deformed bars in tension mainly follows the relevant provisions of the ACI 318-05 specification. The ACI 318-19 specification also uses the same formula, which provides that in addition to raising the upper limit of the applicable longitudinal bar strength from 420 MPa to 690 MPa, also adds the coefficient  $\Psi_g$  of reinforcement strength grade. The development length of rebar in tension,  $l_d$ , is shown in Eq. (1).

$$L_d = 0.9 \frac{f_y}{\lambda \sqrt{f'_c}} \left( \frac{\Psi_i \Psi_e \Psi_s \Psi_g}{c_b + K_{tr}} \right) d_b, K_{tr} = \frac{40A_{tr}}{sn}, 1.0 \leq \left( \frac{c_b + K_{tr}}{d_b} \right) \leq 2.5 \quad (1)$$

According to the relevant provisions of the ACI 318-19 code, the coefficient  $\Psi_g$  for yield strengths of 420 MPa (60 ksi), 550 MPa (80 ksi), and 690 MPa (100 ksi) are 1.0, 1.15, and 1.3, respectively. The revision of this coefficient is based on the research of Orangun *et al.* (1977) and Canbay and Frosch (2005). From their conclusions, it is known that “the splice strength of reinforcement is linearly related to the parameter of splice length  $\sqrt{l_s/d_b}$ ,” and this was based on the rebar lapped test without confinement. As for the development length of confined longitudinal reinforcement in beam members, the conclusions still need to be further verified. In addition, the development increment of the splice strength of rebar decreases with the increase in the lap length; however, the increase in the splice length is not only related to the strength of the rebar. Although increasing the strength of the rebar requires a longer splice length, it is also possible to shorten the lap length requirement by increasing the strength of the concrete.

## Experimental Plan

Figure 1 schematically depicts the bond specimen and test setup used to explore the relationship between the tensile strength and strength grades of reinforcement.

This test setup was intended to simulate the actual mechanism of bonded straight reinforcement in structural flexural members combining moment and shear forces.

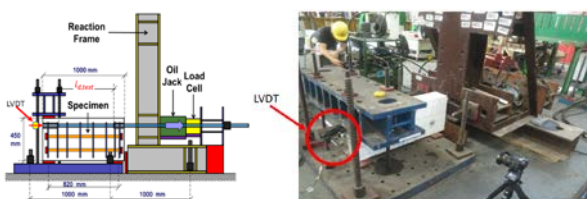


Fig. 1. Test setup.

Each specimen used a single #10 tensile bar with three strength grades (420, 550, 690 MPa) and five concrete strengths (28, 49, 70, 85, 100 MPa). Two  $(c_b + K_{tr})/d_b$  splitting indices of values 3.87 and 4.92 were used for all specimens. A monotonically increasing tensile load was manually applied to each specimen using the oil jack. Each load increment was 5% of the nominal yield strength of the tested reinforcement.

## Results and discussion

The test results show that when the ratio of the average rib height to the average rib spacing of the deformed bar was not less than 0.087 and the splitting index was not less than 3.87, the straight development length of the rebar calculated using ACI 318-14 or ACI 318-19 without considering the coefficient  $\Psi_g$  of reinforcement strength grade (i.e.,  $\Psi_g = 1.0$ ) is still quite conservative. In addition, four failure modes were identified, namely concrete splitting, bar fracture, splice fracture, and rod fracture, as portrayed respectively in Figure 2.



Fig. 2. Failure modes.

The ratios of test bond stress are expressed as  $R_{u14,unl}$  and  $R_{u19,unl}$  without the upper limit of the concrete strength of 70 MPa and the splitting index value of 2.5 stipulated in specification ACI 318-14 and -19 for development length, as shown in Figure 3. The figure on the left shows that the average values of  $R_{u14,unl}$  corresponding to the three steel strength grades are 1.10, 1.05, and 1.12, which are approximately equal. This is a conservative result. The figure on the right shows that the average value of the test bond stress ratio  $R_{u19,unl}$  are 1.10, 1.21, and 1.46 for the three grades (420, 550, and 690 MPa), respectively. Based on the ratio  $R_{u19,unl}$  of the 420 strength grade, the average bond stresses of grades 550 and 690 are 1.1 and 1.33, respectively. This ratio roughly corresponds to the coefficient  $\Psi_g$  in Eq. (1), showing that this formula is already sufficiently conservative, and therefore, there is no need to add the coefficient  $\Psi_g$ .

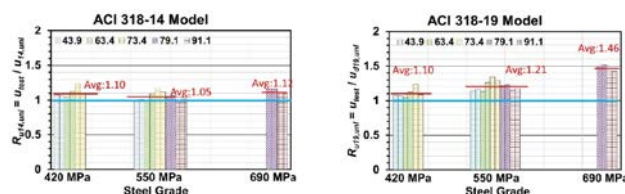


Fig. 3. Bond stress ratio without considering the upper limit of the splitting index and concrete strength.

# Nonlinear Dynamic Analysis Research Based on the TEASPA Fiber Section Model

*Te-Kuang Chow, Associate Technologist, NCREE*  
*Yeong-Kae Yeh, Research Fellow, NCREE*

We replaced a nonlinear hinge with a fiber section using Perform 3D software in order to explore and simulate the backbone curve of the Taiwanese earthquake assessment for structures by pushover analysis (TEASPA) reinforced concrete component. This work was also associated with the function of the mutual control of axial forces and bending moments and the free setting of hysteresis loop rules. Results of analysis of the fiber section model and the nonlinear hinge model were compared for the three-story and seven-story structures of the South Laboratory of the National Center for Research on Earthquake Engineering. The results demonstrated that the nonlinear dynamic analysis of the overall structure based on the fiber section model was both an efficient and feasible approach. A comparison of the experimental values obtained for the nonlinear hinges and fiber section models indicated that the fiber section model could reflect the axial force and bending moment of PM interaction. The convergence of the descent for the fiber section model was better than for the nonlinear hinge model, so the former can be used for nonlinear dynamic analysis. This means that the fiber section can make up for the shortcomings of PM non-linear hinges in ETABS software that cannot freely set hysteresis behavior. The results of analysis of the three-story and seven-story structures based on the fiber section model obtained using the TEASPA method show that the experimental values have a certain degree of accuracy.

The analysis results show that the nonlinear dynamic analysis of the overall structure based on the fiber section model is efficient and feasible. It is compared that the experimental values, nonlinear hinges and fiber section models with each other. The results explain that fiber section model can reflect the axial force and bending moment of PM interaction. The convergence of the descending for fiber section model is better than the nonlinear hinge model. It can be use in nonlinear dynamic

analysis. It means that the fiber section can make up for the shortcomings of PM non-linear hinges in ETABS that cannot freely set hysteresis behavior. Based on the fiber section model obtained by TEASPA method, the analysis results of the three-story and seven-story structures show that there is a certain degree of accuracy with the experimental values. The A-Column and B- Column, as well as the 3 and 7-story buildings can be seen in Figures 1 to 4

The following conclusions were drawn from the results of the comprehensive analysis:

1. The fiber cross-section model ratio of the single-column specimen is quite close to that of the test data, regardless of whether the spine curve or the hysteresis loop is used.
2. The results of the non-linear hinge model and the fiber section model of the three- and seven-story structures are similar for medium and small earthquakes. When a large near-field earthquake was applied, however, the results began to show obvious differences. The results for both models remained similar to the test data.
3. The bending moment strength of the fiber section and the PM nonlinear hinge can change in response to a change in the axial pressure, whereas the nonlinear hinge cannot change. The fiber section model can accurately simulate the PM nonlinear hinge, and the dynamic hysteresis of structural elements can be set freely. Indeed, the behavior reflects the changes in the axial force of the components for the duration of the earthquake and can be used in the seismic evaluation program for weak ground-floor buildings and mid-to-high-rise buildings.

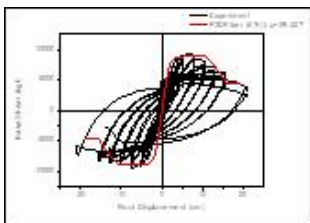


Fig. 1

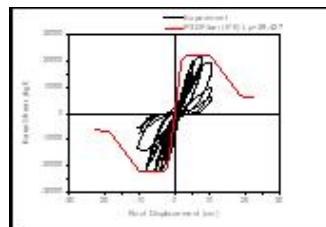


Fig. 2

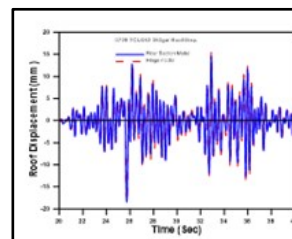


Fig. 3

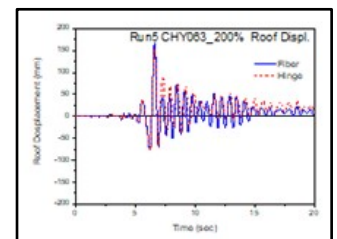


Fig. 4

# Memorandum of Understanding on the Taiwan Structural Monitoring System

Jui-Liang Lin, Research Fellow, NCREE

The research work of Taiwan’s National Center for Research on Earthquake Engineering (NCREE) focuses on three aspects: (1) seismic design, evaluation, and retrofitting; (2) earthquake disaster simulation and seismic risk assessment; and (3) structural monitoring and early disaster warning. The research aims to develop a seismic-resistant and sustainable homeland. For structural monitoring and early disaster warning, the NCREE has recently completed the platform of the “Taiwan Structural Monitoring System”, through which structural monitoring data are more accessible for utilization. Moreover, the NCREE signed a memorandum of understanding (MOU) with the Seismological Center of the Central Weather Bureau (CWB) on April 27, 2022. In that MOU, both sides reached consensus on the scope of cooperation, including transfer and maintenance of monitoring data, privilege and obligation on monitoring data and equipment of monitoring stations, and the management of the Taiwan Structural Monitoring System. On April 27, NCREE Director Chou and colleagues visited Director-General Cheng and Director Chen of CWB (Fig. 1). Fig. 2 shows the moment of signing of the MOU.

Currently, only monitoring data of the office building of the NCREE have been publicized on the Taiwan Structural Monitoring System. In the future, four more building monitoring stations set by the NCREE, 44 building monitoring stations set by the CWB, and bridge monitoring stations will be collectively maintained by the NCREE through the platform. In addition, the monitoring data will be classified and publicized according to the wishes of the building owners. Table 1 shows the five monitoring stations constructed by the NCREE and thirty operating monitoring stations constructed by the CWB.

The data obtained from structural monitoring can be applied to the development of techniques for system identification, structural health monitoring, structural analysis and simulation, rapid evaluation of post-earthquake structural safety, and earthquake early warning. Through research projects and industry–academia collaboration supervised by the Ministry of Science and Technology, the NCREE will devote itself to the

application of the Taiwan Structural Monitoring System.

Table 1. Building monitoring stations maintained by the NCREE.

No	Building Name	No	Building Name
1	NCREE	19	Taipei City hall
2	Administration Bldg., CTSP	20	Taipower Bldg.
3	Tien Chia-Ping Photonic Bldg., NCTU	21	Civil Eng. Bldg., NTUST
4	Dormitory, NCTU	22	Complex, NTUST
5	Computer Center, NCTU	23	Shin Kong Depart. Store, Taipei
6	General Services, NCKU	24	CWB
7	Seismo Graduate School, NCCU	25	Taipei 101
8	An apartment bldg., Tainan	26	Civil Eng. Bldg., NTU
9	Liming Primary School, Chiayi County	27	Library, NCU
10	Southern Region Weather Center	28	CPC Corporation bldg., Miaoli
11	Hualien Tzu Chi Hospital	29	An apartment bldg., Miaoli
12	College of Humanities and Social Sciences, NDHU	30	Taoyuan City hall
13	Emergency Department, Hualien Tzu Chi Hospital	31	Dormitory, NCTU
14	Chang-Gu World Trade Tower, Kaohsiung	32	Library, NCTU
15	Pingtung County Fire Bureau	33	Chienkuo Technology University
16	Kaohsiung City Fire Bureau	34	Taitung Hospital
17	Institute of Earth Sciences, Academia Sinica	35	Taitung Fire Bureau
18	An office bldg., Taipei		



Fig.1. Director Chou and NCREE colleagues visiting Director-General Cheng and Director Chen of CWB.



Fig.2. Signing the MOU.

## EEWS Industry Innovation Integration Workshop at Secutech 2022

*Ching-Hsien Huang, Deputy Technician, NCREE*

Secutech 2022 was held over April 27–29 in Hall 2 of the Taipei Nangang Exhibition Centre. It mainly covered technological development, security, and public safety. Through the exhibition, one could grasp the latest industry trends and comprehensive smart application solutions. Being affected by the COVID-19 pandemic, the number of exhibitors and visitors decreased significantly compared with previous years, but there were still 220 exhibitors. The exhibition unit invited the National Center for Research on Earthquake Engineering (NCREE) and three disaster-prevention application companies to set up an area for the Hybrid Earthquake Early Warning Service (EEWS) to strengthen topics related to earthquakes and people's livelihood applications, and to hold a workshop on the innovation and integration of the hybrid earthquake early warning industry.

The EEWS Area is an introduction to the principles and application of Hybrid EEWS, which is combined with on-site EEWS of the NCREE and the regional EEWS of the Central Weather Bureau. It provides earthquake early-warning information to various regions through the EEWS platform, and devices that receive the hybrid EEWS message can automatically and immediately act upon receiving the warning message to reduce the occurrence of disasters. In this area, in the past few years, we have displayed application products such as early-warning alarm devices, portable earthquake-warning alarm speakers, and other devices, and also presented a demonstration case in Taiwan. In addition, three exhibitors were invited to exhibit together. Sanlien Technology offers EEWS services, vibration measuring, and structural monitoring solutions. P-Waver Inc. focuses on customized earthquake early-warning services, mainly for high-tech factories. The Sigmua D.P.T. Company provides integrated services such as smart homes by combining security systems at homes, schools, and offices.

On the morning of April 29th, we held the Hybrid EEWS Industry Innovation Integration Workshop. There were six speakers and forty attendees at this workshop. NCREE members explained the principles and application and demonstrated the disaster prevention application of Hybrid EEWS in various fields. Five manufacturers were also invited to introduce their products and services for EEWS: Sanlien Technology, P-Waver Inc., Sigmua D.P.T. Company, Kiwi Technology, and Xile Technology (iDaka). Through their presentations, we learned how Hybrid EEWS can be used in daily life to provide disaster-prevention services.

During the exhibition, we contacted people from different industries such as electromechanics, networking, Internet of Things development, telecommunications, technology factories, and others to inquire about the development of Hybrid EEWS, understand the potential and industrial applications, and propose solutions and

advice according to different needs. We hope to provide opportunities for industry matching, continue to carry out publicity and exchanges in the future, expand the application of EEWS, and promote the industry of earthquake disaster prevention.



Fig. 1. Exhibition photographs.



Fig.2. Group photograph with seminar reporters.



Fig.3. Seminar presenters.

# The 8th Asia Conference on Earthquake Engineering (8ACEE)

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The Asia Conference on Earthquake Engineering (ACEE) was initiated by the Association of Structural Engineers of the Philippines, Inc. (ASEP) in 2004 to provide a forum for academics, researchers, and other professionals involved in various aspects of earthquake engineering to share their research experiences.

The first two conferences were held in the Philippines and were attended by hundreds of participants, mostly from the host country. The third ACEE was held in Bangkok, Thailand, on December 1-3, 2010 and was co-organized by four organizations: the Asian Institute of Technology (AIT), the Center for Urban Earthquake Engineering (CUEE) of the Tokyo Institute of Technology, the Engineering Institute of Thailand (EIT), and the Earthquake Engineering Society of Korea. During the third ACEE, nearly 100 papers were delivered by scientists and engineers from more than 20 countries and there were nearly 200 conferees.

The 4th ACEE was jointly held with the 9th International Conference on Urban Earthquake Engineering (9CUEE), and was called the 9CUEE and 4ACEE Joint Conference and was organized by the CUEE. Through the successful combination of 9CUEE and 4ACEE, approximately 300 papers were presented during the conference.

The 5th Asia Conference on Earthquake Engineering was held in Taipei, Taiwan, on October 16-18, 2014. This conference was jointly organized by the National Center for Research on Earthquake Engineering (NCREE) and the National Taiwan University. Nearly 190 papers were delivered by participants from more than 16 countries and

there were nearly 300 conferees. The 6th ACEE returned to the Philippines and was held in Cebu City on the occasion of the 55th anniversary of the ASEP and in commemoration of the 3rd anniversary of the 2013 Bohol earthquake. The 7th ACEE was held in Bangkok, Thailand, on November 22-24, 2018 and was jointly organized by the AIT and the EIT in collaboration with the ASEP.

The 8th Asia Conference on Earthquake Engineering (8ACEE) will be held in Taipei, Taiwan, ROC, during the period November 9-11, 2022, with the theme “Earthquake-Resilient and Sustainable Communities”. 8ACEE is being jointly organized by the NCREE, the Chinese Society of Structural Engineering, the Chinese Taiwan Society for Earthquake Engineering, the Chinese Institute of Civil and Hydraulic Engineering, and the National Science and Technology Center for Disaster Reduction.

8ACEE has been approved and sponsored by the Ministry of Science and Technology, Taiwan, ROC. Due to the COVID-19 pandemic, 8ACEE is being organized as a hybrid conference comprising on-site and online events. The venue for the on-site meetings is the office building of the NCREE. 8ACEE aims to provide an excellent forum to bring together researchers, professionals, engineers, and academics to promote and exchange new ideas and experiences in the fields of earthquake engineering, disaster management, seismic risk reduction, and many other related research fields. Table 1 shows the updated important dates of the conference. For further details please visit the conference website (<http://8ACEE.ncree.org.tw>) (Fig. 1).

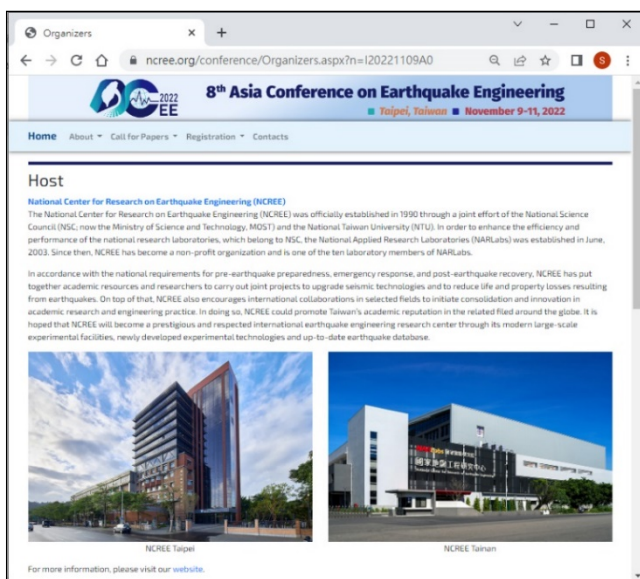


Fig. 1. Website of 8ACEE

Table 1. Important dates relating to 8ACEE

Event	Date
Open date for registration and paper submission	25 February, 2022
Deadline for paper submission	15 July, 2022 (extended)
Author notification of acceptance	31 July, 2022 (extended)
Deadline for revised paper submission	31 August, 2022
Deadline for registration	15 September, 2022
Conference	9-11 November, 2022

## Kaohsiung City 2022 National Defense Mobilization and Disaster Prevention and Rescue (Min'an No. 8) Drill

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### Seismic Performance of TC-BRB

In February, 2022, Russia invaded Ukraine, causing many casualties. Taiwan is also facing an enemy threat. For this reason, the Public Works Bureau of the Kaohsiung City Government held the Min'an No. 8 drill at the Kaohsiung Refinery on March 31, 2022 (Figures 1 and 2). In the past, this event mainly focused on simulating typhoons and earthquakes, but this time a "War Disaster Scenario" was added, which includes simulating scenarios such as missile attacks on residential buildings and rehearsing war disaster rescues and rescues of a large number of casualties. Hundreds of emergency response vehicles and rescue personnel were mobilized in this drill. The drills include war mobilization, protection of infrastructure, rescue of victims and injured, rescue following an aviation accident, debris flow evacuation drills, industrial pipeline disaster drills, and power restoration.

The team is mainly responsible for the project "Search and rescue of human life when buildings collapsed, cross-regional support energy integration and operation". Strong earthquakes reported in various districts of Kaohsiung City were considered, including the collapse of the building at No. 119 Tuku 1st Rd, in which people were trapped under rubble. The department concerned needs to be able to conduct rescues effectively in the golden 72 hours after a disaster, and the emergency response center should collaborate with Yunlin and Chiayi Counties to provide cross-district assistance.

In order to understand the situation at a collapse site and services required for people needing to be rescued, and to confirm whether emergency response vehicles have arrived at the site, the National Center for Research on Earthquake Engineering (NCREE) uses smart drones to conduct damage investigations (Figure 3). Smart drones transmit high-resolution images of the damaged area in real time and thermal infrared images to the 5D Smart City Disaster Prevention & Relief Platform (including a 3D building model) (Figure 4). The images can be displayed on a big screen to identify the positions of trapped people and the situation of the building. This platform can assist the command center to understand the actual situation of the damaged area, formulate strategies for rescue, and then reduce the number of casualties.

The equipment and manpower provided by NCREE include (1) smart drones, (2) the 5D Smart City Disaster Prevention & Relief Platform, (3) the RTK base station, (4) GPS high-precision positioning sensors, (5) 3D aerial photogrammetry modeling of the damaged area, and (6) two platform operators and four drone operators.



Fig. 1. NCREE participated in Min'an No. 8 drill. The picture shows the Mayor of Kaohsiung, Chen Chi-Mai.



Fig. 2. Smart City Group of the NCREE.



Fig. 3. NCREE using smart drones to conduct damage investigations.



Fig. 4. Smart drones investigate the damaged area and transmit high-resolution images to the 5D Smart City Disaster Prevention & Relief Platform.