出國報告（出國類別：國際會議）

出席 **SEEBUS 2011** 國際學術會議
心得報告

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摘要

SEEBUS 是台日韓三國研究建築結構地震工程學者專家之專題討論會議，每年舉辦一次由台日韓三國每年輪流舉辦，第 13 屆於韓國首爾大學舉行。報告人自 2004 年起已連續參加 7 屆，本年度除了於會議發表論文之外，還藉由參與討論瞭解日韓學者之最新研究近況，吸收新知，作為後續研究方向之參考。SEEBUS 研討會是非常好的交流平台，與會成員中包含老中青，資淺的研究人員總是盡力提升自己的研究成果能見度，並與外國教授建立友誼，洽談後續國際合作與互訪交流之可能。報告人回國後仍持續與韓國首爾大學姜炫求教授保持聯絡，對於後續交換研究資訊奠定基礎。

目次

目的

本研討會基本上是定位於交流研討，而確實也達到此目標。然而討論踴躍，問題討論時間常常不夠，建議應可延長此部分時間。SEEBUS 是屬於限制參加人數與討論議題的專題討論會，參加這類專題討論對於年輕的研究人才非常有幫助，這種小型專題討論會反而更有實質深度與時間，讓與會人員可以熟識與交流。

報告人學術專長為混凝土結構，近期擬研究使用高強度鋼筋混凝土之梁柱建築構架耐震行為，俾利未來相關設計方法修訂之參據。此行赴韓國首爾出席 SEEBUS 2011 第 13 屆台灣-韓國-日本建築結構地震工程研討會，主要目的為學術交流蒐集日韓專家意見，並提高我方研究成果之能見度。

過程

11 月 10 日星期四 搭機前往韓國首爾大學。

11 月 11 日星期五 出席 SEEBUS 2011 研討會，李宏仁於會議中第一位簡報論文，詳附件；
論文發表可蒐集專家意見作為調整研究方向及論文修改之參考；
第一天總計參加 4 個時段專題討論，聆聽台日韓專家演講並參與討論；
參加晚宴深化出席會員之友誼。

11 月 12 日星期六 上午出席 SEEBUS 2011 研討會，參加 2 個時段專題討論；
下午工程參觀興建中之 Lotte Super Tower (555m 高，112 層樓)，了解韓國高樓建築發展近況

11 月 13 日星期日 搭機返國

出席學者合照

心得及建議

1. 參訪韓國大學的軟硬體建設已明顯超越我國，韓國自 1990 年起執行的 Brain Korea 21 program 執行績效卓著，足堪本校教學單位持續改進之借鏡。
2. 參加 SEEBUS 研討會，對於瞭解亞太地區建築地震工程學之最新研究趨勢及學術人際交流均亟有幫助，有助於提升台灣學術研究成果於之國際能見度。
3. 本次會議係由本單位校內經費勻支，報告人方得以順利成行。本研討會基本上是定位於交流研討，而確實也達到此目標。SEEBUS 是屬於邀請會員制之專題討論會，參加這類專題討論對於年輕的研究人才非常有幫助，這種小型專題討論會反而更有實質深度與時間，讓與會人員可以熟識與交流。建議我國政府及持續助國內學人出席相關技術交流會議，將國內研究成果國際化，提升能見度。

PRELIMINARY DESIGN RECOMMENDATIONS FOR RC BEAM-COLUMN JOINTS WITH HIGH-STRENGTH REINFORCEMENT

Hung-Jen LEE[1], and Yin-Ru LIN[2]

SUMMARY

This paper presents a database of 110 beam-column joint specimens which were tested by Japanese researchers in the past 20 years. Available test data with high-strength reinforcement (above SD490) were collected and re-organized for investigation in an ongoing Taiwan New RC project at NCREE and a task group in ACI committee 352. Empirical parameter study of the database showed that current ACI design recommendations can be extended to include the use of high-strength reinforcement with modified requirements of joint effective depth, transverse reinforcement, and development lengths. Based on available database investigation, we preliminarily conclude: (1) Story yield drifts of beam-column joint with high-strength reinforcement can also be well estimated using a prior formula for normal-strength materials; (2) Maximum tensile reinforcing ratio of a beam section could be the smaller of 0.025 and $1.5\sqrt{f_c'}/f_y$ in MPa to preclude joint failure without beam yielding; (3) Joint failure modes, story shear strengths, and ductility could be estimated using a proposed empirical strength-to-ductility model derived from the database with conditions of bond and transverse reinforcing index. Provided information could be referred for further updating the design recommendations of beam-column joints for high-strength reinforcement.

***Keywords**: ductility; high-strength reinforcement; joint; shear strength.*

INTRODUCTION

Since the New RC project in Japan, there have been many tests conducted by Japanese researchers to validate the use of high-strength reinforcement and concrete. Borrowing the successful experience of New RC Project, Taiwanese researchers would like to push another New RC Project in Taiwan. The project is aim to promote the use of high-strength reinforcement in concrete structures and upgrade the construction technologies in Taiwan.

This paper presents a preliminary study on the shear strength and ductility of beam-column joints with high-strength reinforcement. A database of 110 beam-column joint specimens were constructed by investigating available papers published in Japan (Abe et al. 2006; Adachi et al. 2006; Hara et al. 2001; Hara et al. 2005; Hori et al. 2006; Hori et al. 2004; Hosoya et al. 2003; Imai et al. 2003; Inoue et al. 2004; Iwaoka et al. 2003; Iwaoka et al. 2005; Kando et al. 1997; Kawazoe et al. 2008; Kimoto et al. 2006; Kiyohara et al. 2005; Kiyohara et al. 2004; Maruta and Sanada 2004; Masuo et al. 2006; Nakachi and Tabata 1995; Nakatani et al. 1999; Nakazawa et al. 2000; Nakazawa et al. 2001; Nakazawa et al. 2003; Nakazawa et al. 2009; Shinjo et al. 2005; Watanabe et al. 2005). To date, the database has test results of 53 interior and 57 exterior beam-column joint specimens, all tested under reverse cyclic lateral loads with constant column axial loads. Those joint specimens with varying column axial loads, transverse beams, or slabs are excluded in this database and to be studied in the near future.

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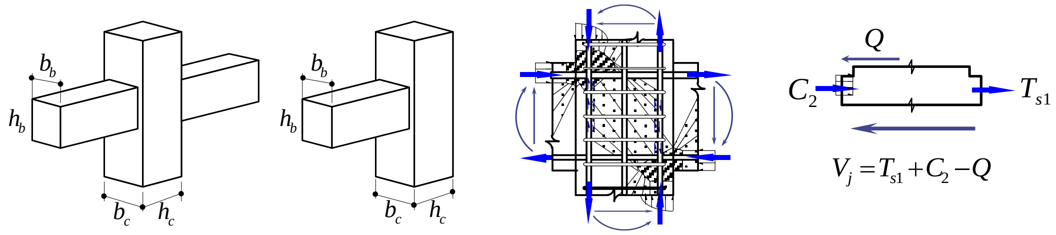
DATABASE CONSTRUCTION

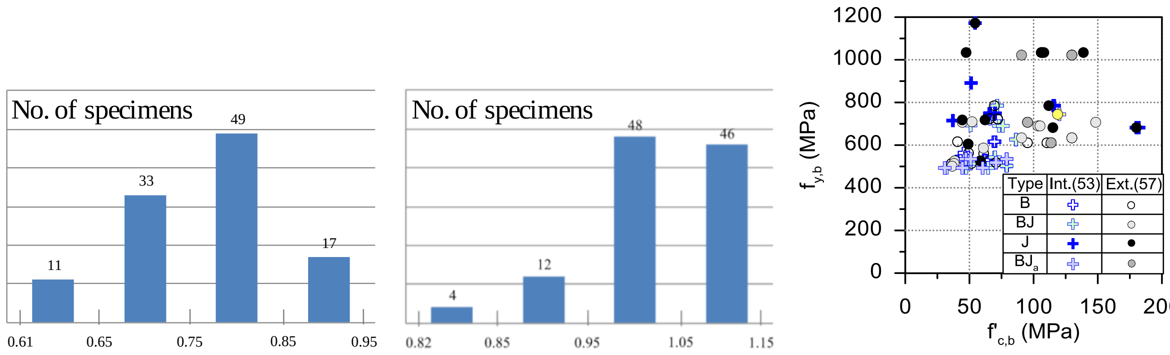
To investigate the seismic performance of beam-column joint with high-strength reinforcement, a database for tests of modern reinforced concrete beam-column joints was constructed by extensively reviewing the papers published in Japan in the last two decades. Typical interior and exterior joints without transverse framing members are included in the database, as shown in Figures 1(a) and 1(b). Each beam was concentric with respect to the column centerline and has a section width not exceed the width of the column. All specimens were beam-column subassemblages isolated from contraflection points of beams and columns, and tested under quasi-static cyclic lateral loading (typical repeated cycles for each drift ratio ranged from one to three) to simulate the earthquake-introduced forces acting on the joints (Figure 1). Equilibrium criteria for reinforced concrete beam-column joints (Paulay 1989) are used to calculate the joint shear force when the adjacent beams are subject to positive or negative moments due to story drifts.



(a) Interior joint (b) Exterior joint (c) Forces acting on a joint (d) Joint shear force

Figure 1. Terminology and forces acting on a joint under lateral loads

Figure 2 shows the range of aspect ratios and material strengths for the database. The beam-to-column width ratios ranged from 0.6 to 0.9, while most of the beam-to-column depth ratios fall around 1.0. The maximum and minimum column depths in the database are 600 mm and 340 mm, most of the column sections are 400-450 mm in square. Small-scaled specimens are not included in the database.



(a) Beam-to-column width ratio b_b/b_c (b) Beam-to-column depth ratio h_b/h_c (c) Beam bar f_y versus f'_c

Figure 2. Range of aspect ratios and material strengths for the database

As shown in Figure 2(c), cruciform and circular symbols represent 53 interior and 57 exterior joints, respectively. The concrete compressive strengths ranged between 40 and 190 MPa, while the bar yield strength of beam longitudinal reinforcement ranged from 490 to 1200 MPa. Only the joint specimens used high-strength reinforcement (SD 490, 590, 685, or 980) are included in the database. Notably, SD 490 reinforcement were used for beam bars with concrete strength not greater than 80 MPa, while SD 685 reinforcement can be used for a much wider range of concrete strength.

Generally, it is more preferable to preclude shear failures and arrange flexural plastic hinges in beams rather than in columns for seismic design of concrete frames. Within the database, beam and column shear failures are precluded by proper detailing and adequate transverse reinforcement. To develop beam hinging or test joint shear strengths, all the database specimens had strong-column and weak-beam arrangements. For interior joints, most of the beam bars are continuously extended through the joints, and some are mechanically spliced in the joints. In the exterior joints, beam longitudinal bars are anchored by hooked or headed bars extended into the joints with an embedment length not less than $0.5\,h_c$ or $10\,d_b$. All specimens had closely-spaced joint transverse reinforcement. It should be noted that crossties are not code-required in Japan for joint transverse reinforcement. Thus, many joint specimens did not have crossties in the joints.

FAILURE MODES AND DUCTILITY

Within the joint database, there are three typical failure modes: (1) B-failure refers to beam flexure failure in the beam plastic hinges while the joint remains elastic; (2) BJ-failure refers to joint shear failure along with yielding of beam reinforcement; (3) J-failure refers to joint shear failure without yielding of beam reinforcement. This classification of failure modes is well-accepted in Japan for development of design guidelines for beam-column joints (Kitayama et al. 1991). Besides three basic failure modes, the database had 11 interior joints and 6 exterior joints of BJa-failure, which refers to anchorage failure of the beam bars in the joint. For interior joints, BJa-failure would exhibit very pinching hysteretic curves with mild strength degradation. However, for exterior joints, BJa-failure may have a sudden drop of lateral resistance due to pullout or side-blow-out of beam bars.

Figure 3 shows the concepts which the joint shear capacity decreases as the inelastic drifts increases. If the joint shear capacity remains greater than the demand to the end, the maximum strength is limited by the beam flexure capacity (B-type failure). When the joint shear capacity falls below the shear demand from beam flexural hinging, the joint will fail in shear before or after beam yielding (BJ-type or J-type). Beam-column joints subjected to inelastic drift reversals often underwent significant bond deterioration along the reinforcing bars from the adjacent beams. As the inelastic drift increases, the joint transverse reinforcement may yield progressively and joint panel concrete may crack excessively, both lead to degrade the joint shear strength. This phenomenon of degradation of joint shear capacity was pointed out by Joh et al. (1991). Analytical models for the degradation of joint shear capacity have been suggested by prior researchers (Hong et al. 2011; Lee et al. 2009). However, these models are quite well for analysis but too complicate for design purpose.

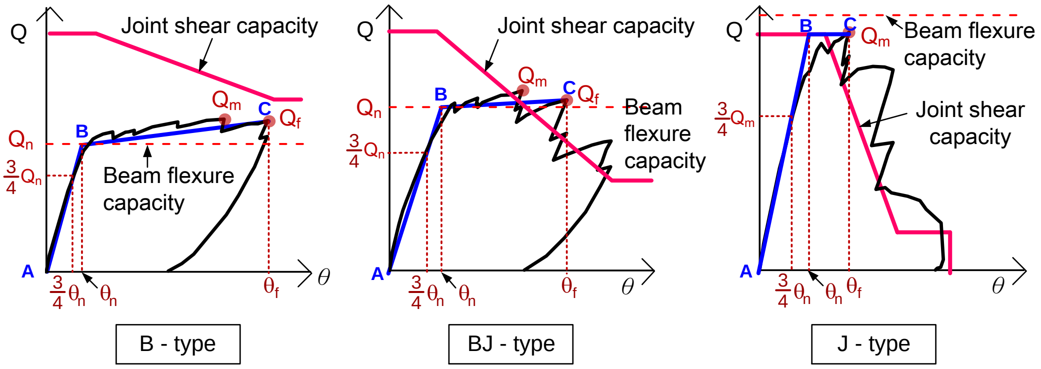


Figure 3. Definitions of yield drifts, failure drifts, and failure modes

The definitions of experimental yield drifts and failure drifts are also illustrated in Figure 3. For each test specimen, the envelope of story shear Q versus story drift θ is extracted from its cyclic load-deformation curves. The idealized elastic branch of line AB crosses the experimental envelope at 75% of nominal story shear force Q_n or maximum story shear Q_m, whichever is smaller, and reaches point B to define the idealized yield drift θ_n. The nominal story shear force Q_n is the theoretical strength corresponding to the development of nominal flexural strengths M_n at beam ends. It should be noted that the failure drifts θ_f may exceed the drift of maximum measured strength Q_m, as shown in Figures 3 (a) and (b). Sometimes, the descending branch or post-strength curves are not available from laboratory testing, due to setup limitations or loading procedures. For consistency of the database, the failure is defined at point C, where the lateral resistance start to drop or fall below Q_n, to determine ductility ratios ($\mu = \theta_f/\theta_n$) of the joint specimens.

PRELIMINARY DESIGN RECOMMENDATIONs

Story Yield Drifts for Moment Frames

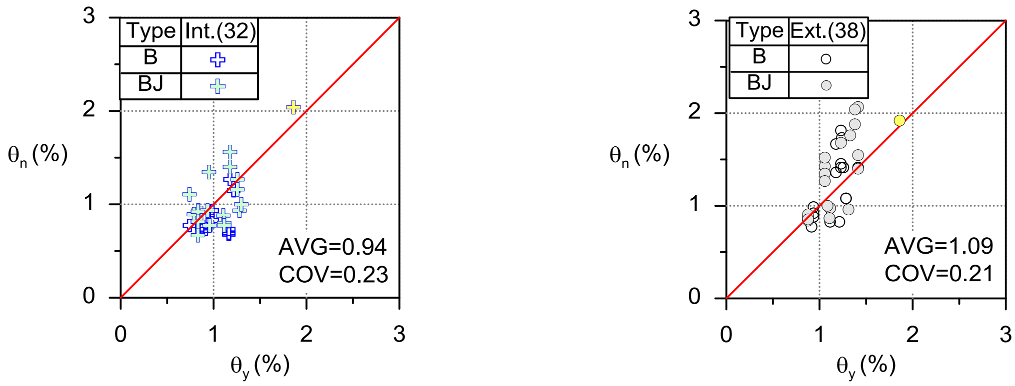
It should be noted that reinforced concrete moment-resisting frames are much more flexible than commonly assumed by designers, even when cracking is taken into account. The use of high-strength reinforced concrete in place of normal strength materials have advantages of smaller member sizes, longer beam span, and wider floor space. Therefore, it would be more important for the designers to estimate the frame stiffness and deflection. Conventional design assumptions would predict yield story drifts about 0.5%, which may be under-estimated for high-strength reinforced concrete frames.

Priestley (1998) has suggested a rational expression for story yield drift θ_y of reinforced concrete moment frames.

$$\theta_y = 0.5\varepsilon_y \frac{l_b}{h_b} \tag{1}$$

where θ_y = story yield drift of the frame; ε_y = yield strain of the beam longitudinal reinforcement; l_b = the length of the beam bay between column centers; and h_b = beam depth. Priestley calibrated this simple expression against numbers of beam-column joint experiments with normal-strength reinforcement, in which the yield story drift ratios ranged from 0.4% to 1.5%, with a mean experiment-to-prediction ratio of 1.01 and 15% coefficient of variation.

Figure 4 compares the experimental and predicted yield drifts for the test data in the database. The correlation between predicted and experimental yield drifts was quite surprisingly well. The experimental yield drifts ranged from 0.7% to 2.1%, where the maximum value of 2.1% was obtained from a large-scale cruciform joint specimen tested at NCREE by the first author. As shown in Figures 4(a) and (b), the mean experiment-to-prediction ratios are 0.94 and 1.09 for interior and exterior joints, respectively, where the coefficients of variation are 23% and 21%.



(a) 32 interior joints with beam yileding (b) 38 exterior joints with beam yileding

Figure 4. Correlations between experimental and theoretical yield drifts

Maximum Reinforcement Ratio for Beams

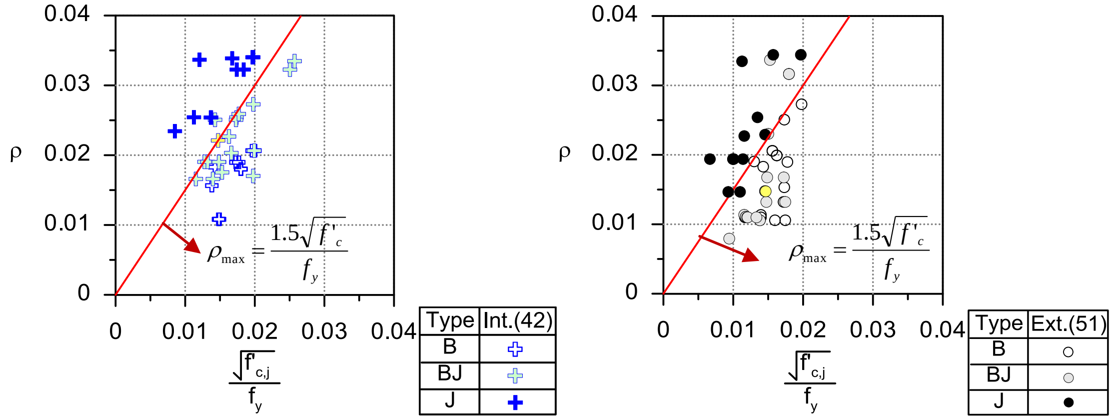
It is an indisputable fact that the Design Codes of Concrete Structures in Taiwan is based on the ACI 318 Code. Our challenge is how to extend or modify current design provisions to include the use of high-strength reinforcement. Based on the database investigation, several recommendations are drawn for preliminary design.

According to the code commentary, the maximum reinforcement ratio of 0.025 for beams is based primarily on considerations of steel congestion and, indirectly, on limiting shear stresses in beams of typical proportions with conventional reinforcement. For frame beams designed using high-strength reinforcement and concrete, the limiting reinforcement ratio of 0.025 is not always proper. Based on limiting joint shear stress generated from beam flexural reinforcement, a modification of the maximum reinforcement ratio is proposed as below.

$$\rho_{\max} = \text{the larger of } \frac{1.5\sqrt{f_c'}}{f_y} \text{ and } 0.025, \text{ where } f_c' \text{ and } f_y \text{ in MPa} \tag{2}$$

Figure 5 illustrates the limiting reinforcement ratio of $1.5\sqrt{f_c'}/f_y$ and failure modes. Within the database, all the 10 J-failure interior joint specimens (solid cruciform symbols) are precluded by this simple expression. On the other hand, 12 of 13 J-failure exterior joint specimens (solid circular symbols) can be precluded. Obviously, the red lines of $1.5\sqrt{f_c'}/f_y$ could effectively separate those J-failure specimens from the others. The upper limit of 0.025 was also kept because of consideration of steel congestion. It should be noted that the limiting reinforcement ratio of $1.5\sqrt{f_c'}/f_y$ is based on the shear strength of a joint without transverse beams. For beams framing into joints with transverse beams, an amplification factor may be applied. Further study is recommended to determine this assumption.

In general, for typical proportions of beams and columns, proposed maximum reinforcement ratio of $1.5\sqrt{f_c'}/f_y$ is relatively conservative and rational. Without limiting reinforcement ratio of $1.5\sqrt{f_c'}/f_y$, there are 1/10 interior joints and 9/13 exterior joints exhibited J-failure, although their reinforcement ratios are less than 0.025. In contrast, several joint specimens with beam reinforcement ratios greater than 0.025 still exhibited BJ-failure or B-failure with adequate ductility.



(a) Interior joints (b) Exterior joints

Figure 5. Maximum tensile reinforcement ratios for beams to preclude J-type failures

Maximum Shear Stresses in Joints

Based on the capacity design concept, the demand of the joint shear force V_u is dominated by the probable flexural moment strength at beam ends. When computing design shear forces, a probable strength of αf_y for the beam longitudinal reinforcement shall be included. Current codes taken α=1.25 for conventional ASTM A706 Grade 420 MPa reinforcing bars. For SD 685 reinforcement, due to similar over strength and higher yield strength, taking α=1.15 should be proper than 1.25.

The current ACI design procedures for estimating joint shear strength are based on recommendations of ACI Committee 352, as the equation shown below.

$$\phi V_n = \phi\gamma\sqrt{f_c'}b_j h_c \geq V_u \tag{3}$$

where ϕ is the strength reduction factor of 0.85; $\gamma\sqrt{f_c'}$ is the nominal joint shear stress of $1.0\sqrt{f_c'}$ MPa for exterior joints without transverse beams, and of $1.25\sqrt{f_c'}$ MPa for interior joints without transverse beams. h_c is the column depth; and b_j is the effective joint width for considering the effects of the column's aspect ratio and joint eccentricity. The design philosophy embodied in Eq. (3) is that during anticipated earthquake-induced loading and inelastic drift reversals, a well-confined joint can resist the design shear forces to attend 2% or 3% story drift.

The effective joint width defined in ACI 318 Code is an out-of-date version. Since all test specimens in the database are concentric beam-column joints, this paper used the basic definition of $b_j = (b_b + b_c)/2$ for concentric joints suggested by (ACI Committee 352 2002) .

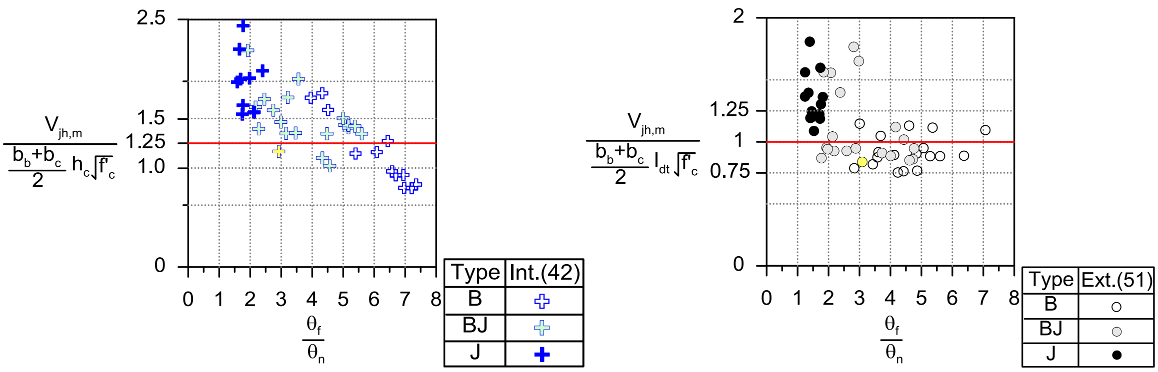
For each test specimens in the database, the maximum shear force $V_{jh,m}$ acting on the joint can be estimated by force equilibrium (Figure 1(d)).

$$V_{jh,m} = T + C - Q_m = Q_m\left[\frac{(l_b - h_c)}{z_b}\times\frac{l_c}{l_b} - 1\right] \tag{4}$$

where Q_m is the maximum story shear measured during testing; z_b is the sectional lever arm and approximately 7/8 or 0.9 of the effective depth of the beam section; l_b is the length of the beam bay between column centers; and l_c is the story height. It is should be noted that $V_{jh,m}$ is the maximum imposed joint shear forces during testing. For J-failure and BJ-failure specimens, $V_{jh,m}$ is limited by joint shear and can be referred as experimental joint shear strength. For B-failure specimens, $V_{jh,m}$ is dominated by the beam flexural capacity and less than the potential joint shear strength.

Figure 6 shows that the experimental joint shear stresses, normalized to effective joint area and square root of f'_c in MPa, decrease as the imposed drift ductility increases, in particular for the shear strengths measured from J-failure and BJ-failure specimens. For interior joint data shown in Figure 6(a), it seems the constant γ =1.25 for Eq.(3) is conservative for estimating joint shear strength at a drift ductility of 3 or 4, without applying strength reduction factor $\phi = 0.85$.

On the other hand, the lower constant γ =1.0 is not conservative for exterior joints in the database. Referred to the Kajima Hi-RC guideline equation (Kato et al. 1991), this paper used the same constant γ =1.0 for exterior joints with a shorter effective joint depth of l_{dt}, which is the anchorage length of beam longitudinal bars terminated in the joint. Applying this reduction, the normalized experimental joint shear stresses are shown in Figure 6(b). It seems the constant γ =1.0 could also be conservative for estimating joint shear strength at a drift ductility of 3 or 4.



(a) Interior joints (b) Exterior joints

Figure 6. Normalized experimental joint shear stresses and ductility ratios

PREDICTION OF JOINT SHEAR STRENGTH AND DUCTILITY

Proposed Model for Degradation of Story Shear Strength with Drift Ductility

It can be also found in Figure 6 that most of the BJ-failure data fall within the shear strength range of $(\gamma \pm 0.25)\sqrt{f'_c}$ and drift ductility range of 2-6. Based on this finding and parameter study of the database, this paper proposed a simple model for degradation of joint shear strength with drift ductility. The proposed degradation model for joint shear strength is expressed as below.

$$V_j = \left\{\gamma - \left[\frac{\mu-(1+\beta)-(1+\lambda)}{(1+\lambda)}\right]\times 0.25\right\}\sqrt{f'_c}A_j \qquad (5)$$

where V_j is joint shear capacity as a function of connection type constant γ, drift ductility μ, bond index β, and tie index λ; A_j is the effective joint area computed from effective joint depth h_j times effective joint width b_j. Effective joint depth h_j shall be the column depth h_c for interior joints and l_{dt} for exterior joints, respectively.

The bond index β is the inverse of the design bond stress along a beam bar in a joint, in terms of $\sqrt{f'_c}$. This design bond stress is derived for the conditions of beam yielding at both faces of the column.

$$\beta = \frac{4}{\left(1+A_{s,bot}/A_{s,top}\right)}\frac{\sqrt{f'_c}}{f_y}\frac{h_j}{d_b} \qquad (6)$$

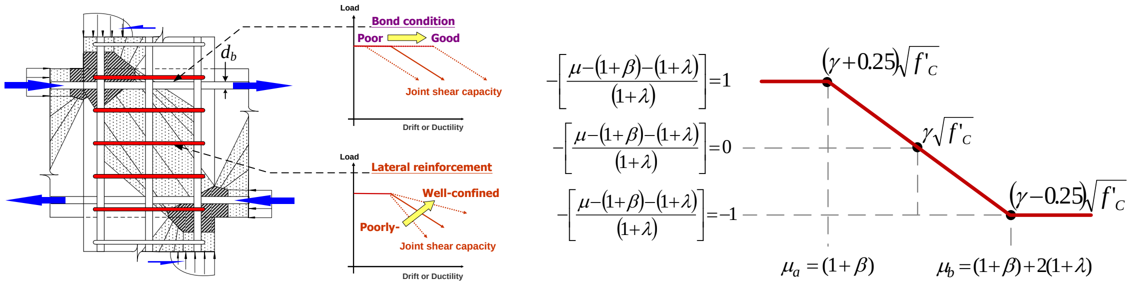
with the limitation of $0 \le \beta \le 1.0$ and $A_{s,bot}/A_{s,top} \le 1.0$, where $A_{s,top}$= area of bottom reinforcement and $A_{s,top}$= area of top reinforcement of the beam. For exterior joints, the bond index β is computed by setting $A_{s,bot}/A_{s,top} = 0$ and h_j=l_{dt}.

The tie index λ is the ratio of maximum tie force in the joint to the tensile force of the beam reinforcement.

$$\lambda = \frac{\Sigma A_{sh}\bar{f}_{yt}}{A_s f_y} \qquad (7)$$

with a limitation of $0 \le \beta \le 1.0$; where ΣA_{sh} is the total cross sectional area in the shear direction of transverse reinforcement within the joint; $\bar{f}_{yt}$ is the yield strength of transverse reinforcement; When computing tie index λ, $\bar{f}_{yt}$ cannot be taken greater than 785 MPa. The maximum yield strength of 785 MPa is tentatively assumed in this paper. It should be a function of concrete strength and reinforcement configuration, which may be improved in the future study.

Figure 7 illustrates the proposed model, which assumes better bond condition (bond index β moves to 1.0) could delay the degradation, while enhance the joint transverse reinforcement (tie index λ moves to 1.0) could reduce the rate of degradation. For poor bond and tied conditions (β=0; λ=0), the joint shear degradation initiates at μ=1 and stops at μ=3. For the excellent bond and tied conditions (β=1; λ=1), the joint shear degradation initiates at μ=2 and stops at μ=6.



(a) Effects of bond and tie indexes (b) Degradtion of joint shear strength

Figure 7. Proposed degradation model for joint shear strength with drift ductility

Figure 8 illustrates the joint failure modes and drift ductility can be predicted by comparing the beam flexural and joint shear capacities. The nominal story shear force Q_n^B is the theoretical B-failure strength corresponding to the development of nominal flexural strengths M_n at beam ends. The nominal story shear force Q_n^J is the back-calculated BJ-failure strength using Eq.(4) and nominal joint strength of $\gamma\sqrt{f_c'}$. The degradation of joint shear capacity can be estimated using Eqs.(5-7). If Q_n^B is greater than the upper bound of joint shear capacity, J-failure is predicted. If Q_n^B fall between the upper and lower bound of the joint shear capacity, BJ-failure is predicted. The failure drifts are computed from ductility times the theoretical yield story drift θ_y given in Eq.(1).

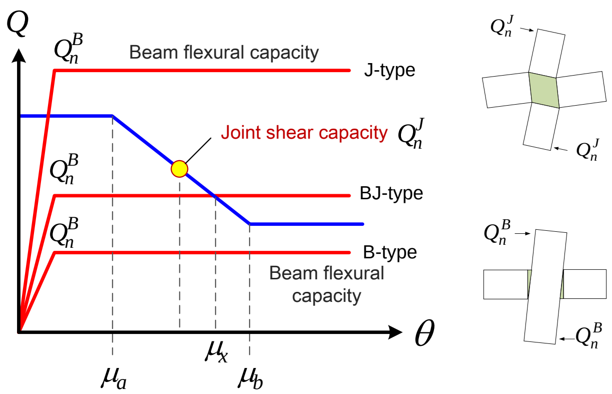


Figure 8. Illustrations for prediction of failure modes and ductility

Experimental Verification

All the test results in the database are used to verify the proposed degradation model. Figures 9 shows several comparisons with test results, where the solid curves are reproduced envelopes of the hysteretic curves of the tested specimens. The flat broken lines denotes the nominal story shear Q_n^B corresponding to the development of nominal flexural strengths M_n at beam ends. The tri-linear degrading broken lines represent the predicted story shear limiting to the degrading joint shear strength with ductility. The test specimens shown in Figure 9 are well-predicted in failure modes, strengths, and drift capacity. Within the database, about 3/4 test specimens can be successfully predicted in correct behavior in strengths and failure modes.

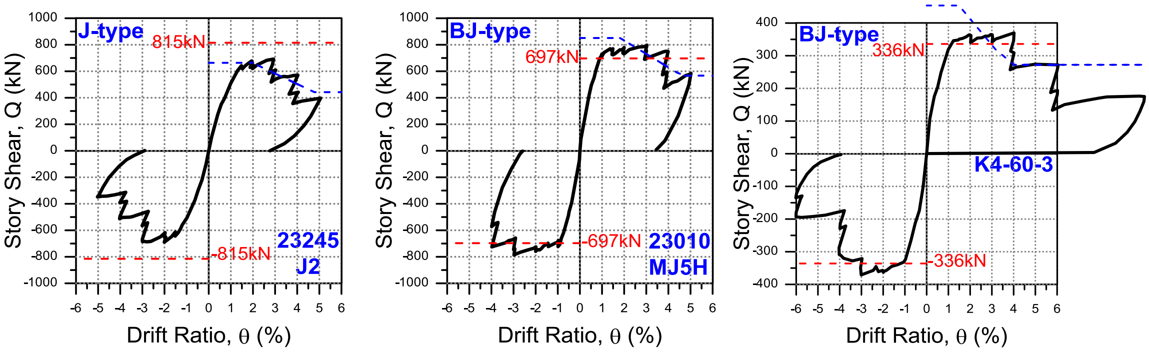


Figure 9. Comparison of proposed degradation model and test results (well-predicted cases)

Figure 10 shows the comparisons of experimental-to-predicted story shear forces and failure modes. The failure mode prediction depends on the ratio of Q_n^J to Q_n^B, which is the horizontal axis in Figure 10. The average experimental-to-predicted story shear strengths are about 1.25 with a coefficient of variation of 0.16. All the test data are at conservative side except two BJ-failure data shown in Figure 10. The predictions of story shear strengths and failure modes are quite reasonable.

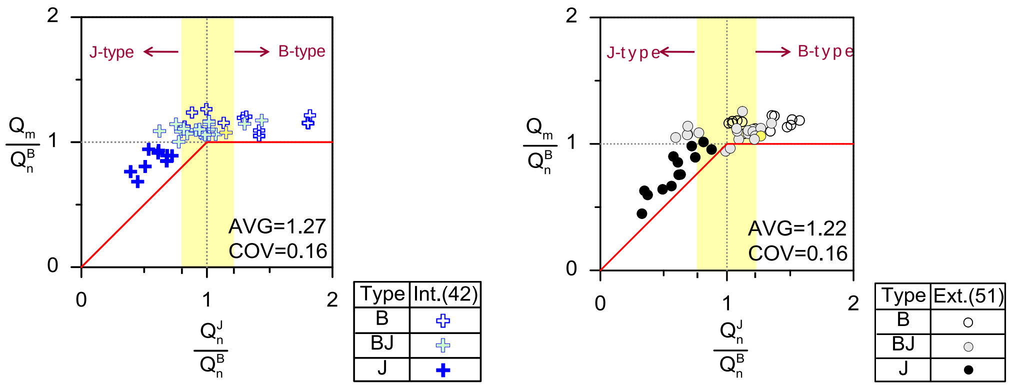


Figure 10. Comparisons of experimental-to-predicted story shear forces and failure modes

Figure 11 shows the correlation between experimental failure drifts and predicted drifts ($\theta_x = \mu\theta_y$). Due to combination of variations from μ and θ_y, it seems relatively scatter. Since the proposed degradation model is simple, this result is still acceptable.

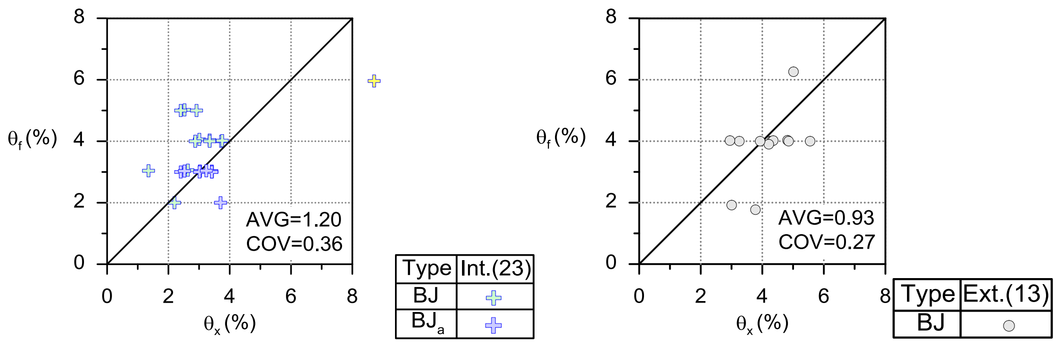


Figure 11. Comparisons of experimental-to-predicted story drifts

CONCLUDING REMARKS

An extensive experimental database of reinforced concrete beam-column joints, made with high-strength reinforcement, subjected to cyclic lateral loading, and experiencing different failure modes has been constructed to review current code limiting values. Several preliminary design recommendations have been drawn from database investigation, and a degradation model of joint shear strength with imposed ductility is also proposed.

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Earthquake Engineering for Building Structures (SEEBUS 2011)
Seoul, Korea, November 11-12, 2011

Preliminary Design Recommendations for RC Beam-Column Joints with High-Strength Reinforcement

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*Associate Professor
National Yunlin University of Science & Technology
TAIWAN

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National Yunlin University of Science & Technology

Outline

- Why we need high-strength reinforcement
- Database investigation
- Prediction of joint strength and ductility
- Preliminary design recommendations

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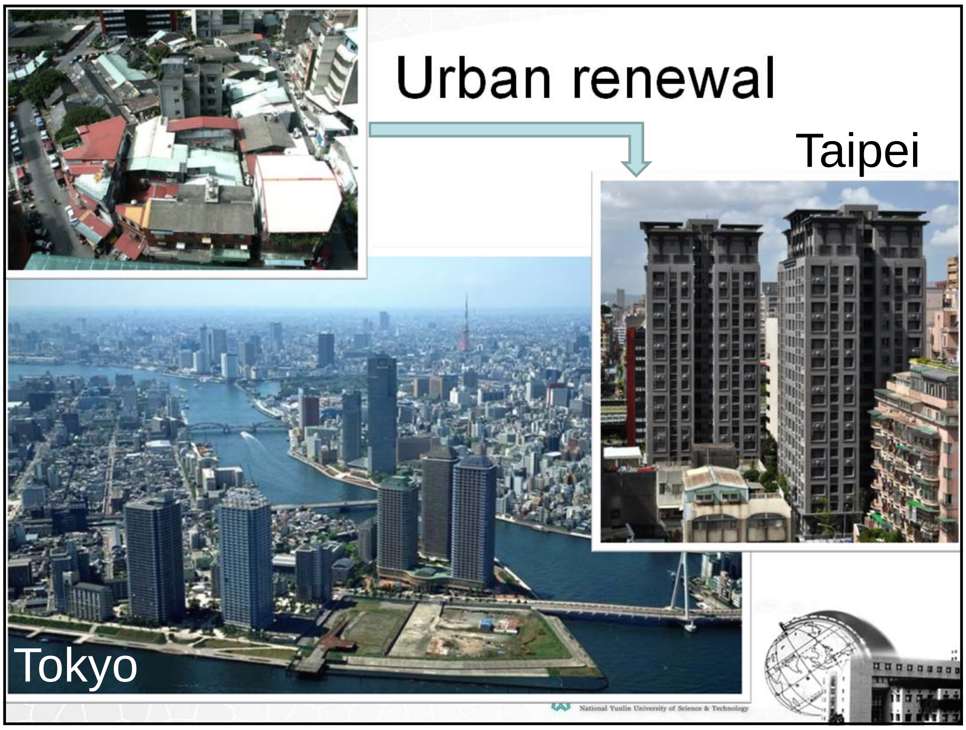
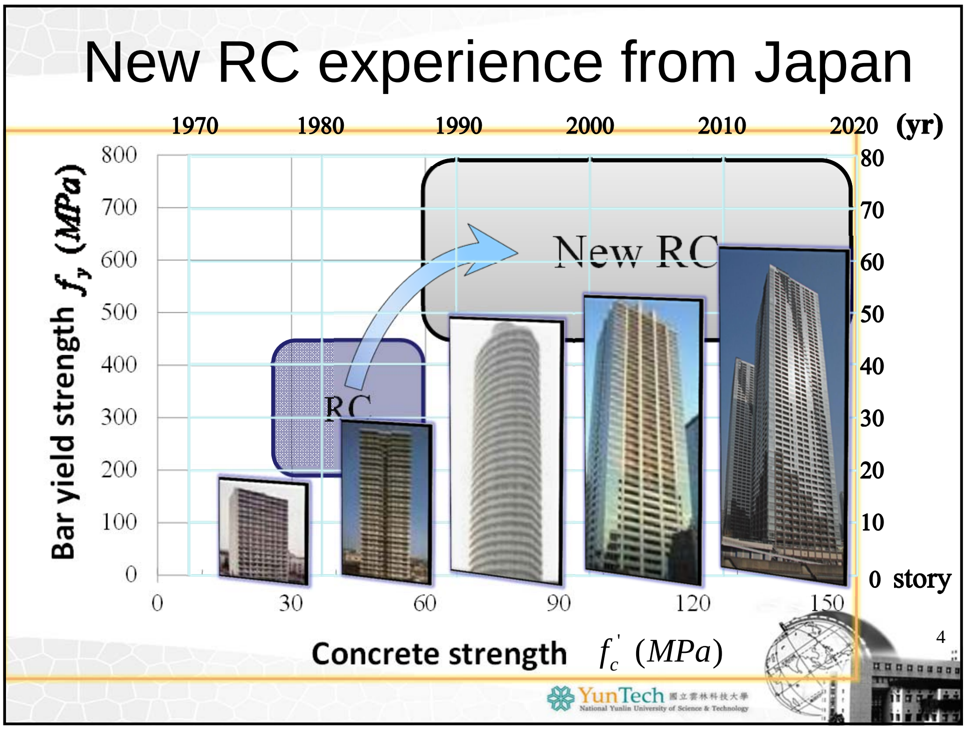


Urban renewal
Taipei
Tokyo
National Yunlin University of Science & Technology



New RC experience from Japan
1970
1980
1990
2000
2010
2020
(yr)
Bar yield strength f_y (MPa)
800
700
600
500
400
300
200
100
0
80
70
60
50
40
30
20
10
0 story
New RC
RC
0
30
60
90
120
150
Concrete strength f_c' (MPa)
4
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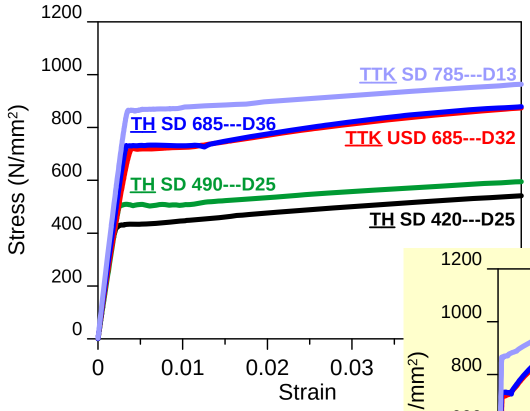
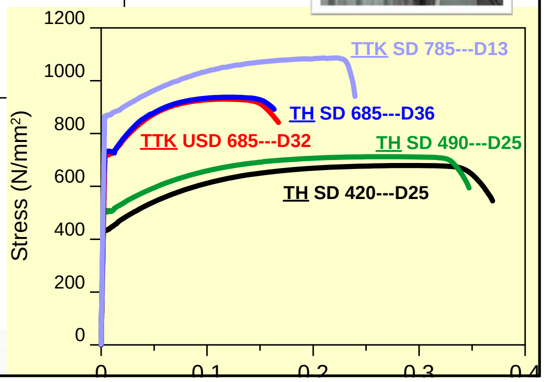
Stress-strain curves of HS bars





Design provisions of Beam-Column Joints

AIJ Design Guidelines

➢Architectural Institute of Japan (AIJ), 1999, Design Guidelines for Earthquake Resistant Reinforced Concrete Buildings Based on Inelastic Displacement Concept, with Commentary, (in Japanese)

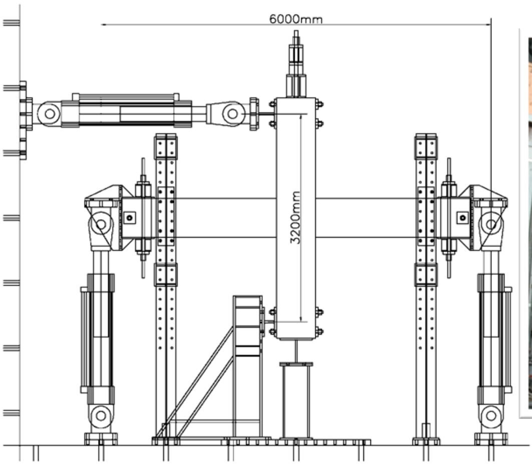
ACI 318 Building Code

➢ACI Committee 318, 2008, Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary (318R-08),

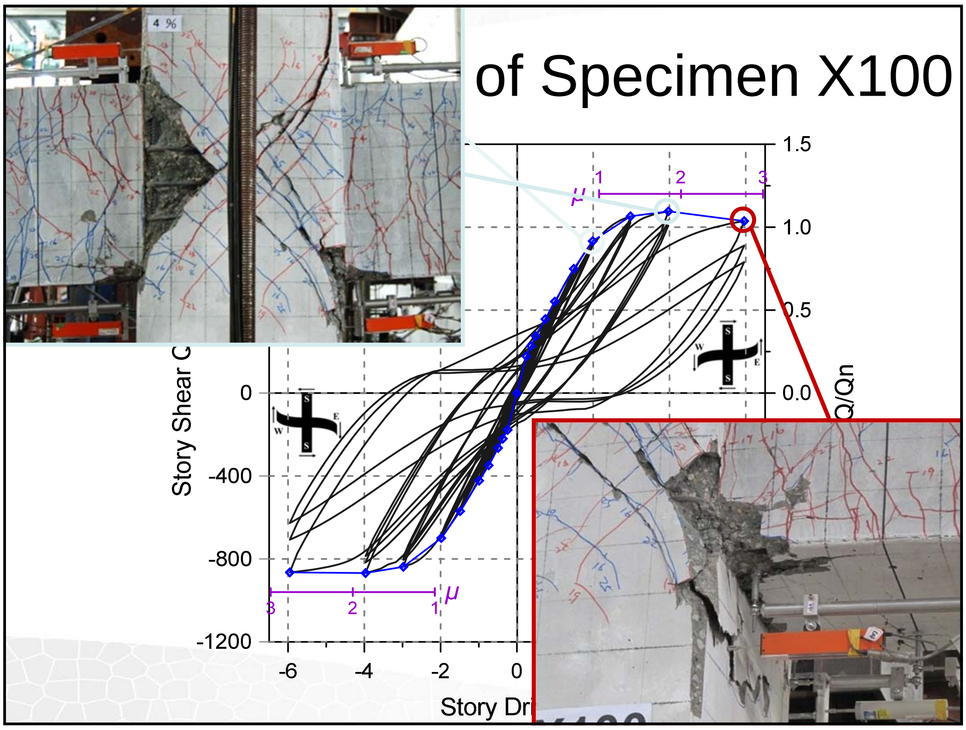
Is it OK to use the ACI method to design beam-column joints with high-strength reinforcement?

- Join shear strength ?
- Transverse reinforcement, Ash
- Bond and anchorage requirements

Experiments at NCREE (2010)



of Specimen X100



Outline

- ~~Why we need high-strength reinforcement~~
- Database investigation
- Prediction of joint strength and ductility
- Preliminary design recommendations

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Reviewing available test data

- Japan

  AIJ 日本建築学会 - 大会学術講演梗概集

- On-line library

  eAINA | CRIC   CiNii NII 論文情報ナビゲータ[サイニィ]   J-STAGE   Google

- Data screening criteria

  High-strength rebars (SD490 and above)

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High-Strength Reinforcement

Grade	Yield strength	Tensile strength	Elongation
	N/mm^2	N/mm^2	(%)
SD490	490~625	≥620	≥12
SD685	685~785	≥860	≥10
SD980	≥980		≥7
SD785	≥785	≥930	≥8
SD1275	≥1275	≥1420	≥7

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Japanese references (1/2)

年代	作　者	題　目	十字型	ト字型
1995	仲地 唯治、田畑 健	高強度材料によるRC柱梁接合部に関する実	6	-
1997	貫洞 覚、斎藤 誠、	高強度RC造中柱梁接合部の梁主筋の付着性	5	-
1999	中谷 庄吾、渡辺 直	機械式に定着された外柱梁接合部の構造性	-	4
2000	中澤 春生、熊谷 仁	超高強度鉄筋コンクリート構造(Fc=120 N/m	-	2
2001	竹内 博幸、岸本 剛	円形定着板を用いた機械式定着工法の開発(	-	3
	原 孝文、鴇田 隆、	高強度材料を用いたRC柱梁接合部架構の構	1	1
	中澤 春生、熊谷 仁	超高強度鉄筋コンクリート構造(Fc=120 N/m	2	1
2003	中澤 春生、黒瀬 行	機械式定着と折曲げ定着を併用したRC 造柱	3	-
	岩岡 信一、堀 伸輔	超高強度鉄筋コンクリート構造の柱梁接合	2	1
	今井 弘、長谷川浩	機械式に定着された梁主筋が外柱梁接合部	-	3
	細矢 博、岸本 剛、	梁主筋の機械式継手が柱梁接合部の構造性	4	-
2004	清原 俊彦、田才 晃	梁主筋を機械式定着した高強度コンクリー	-	5
	堀 伸輔、岩岡 信一	超高強度鉄筋コンクリート構造の柱梁接合	3	1
	井上 寿也、益尾	機械式定着工法によるRC 外柱梁部分架構の	-	6

接下頁↓

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Japanese references (2/2)

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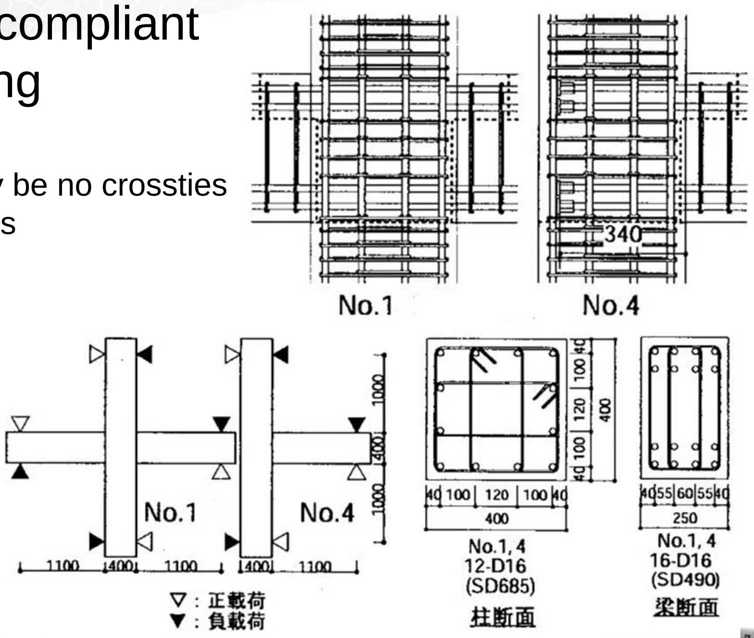
年 代	作　　者	題　　目	十 字 型	ト 字 型
2004	丸田 誠、真田 暁子	170N/mm2 を超える高強度コンクリートを	3	-
2005	清原 俊彦、長谷川	梁主筋を機械式定着した高強度鉄筋コンク	-	7
	原 孝文、渡辺 英義	超高強度コンクリートと高強度鉄筋を用い	5	2
	岩岡 信一、堀 伸輔	超高強度鉄筋コンクリート構造の柱梁接合	2	2
	新上 浩、小坂 英之	柱梁接合部内に機械式継手を用いたRC造架	4	-
2006	阿部 洋、山下真吾	高強度鉄筋SD590 を用いた柱・梁接合部に	3	-
	益尾 潔、足立 将人	梁主筋USD590を機械式定着したRC造ト形	-	4
	足立 将人、益尾 潔	梁主筋USD590を機械式定着したRC造ト形	-	4
	木本 敏一、中岡 章	柱梁接合部プレキャスト架構の加力実験（	3	3
	堀 伸輔、岩岡 信一	超高強度鉄筋コンクリート構造の柱梁接合	4	-
2008	川添由喜子、諸伏	機械式定着を用いた高強度梁主筋コンクリ	-	6
2009	中澤 春生、大久保	超高強度鉄筋コンクリート構造（Fc=180N/	2	1
2010	李宏仁、郭青翰	新高強度鋼筋混凝土梁柱接頭耐震性能研究	1	1

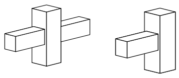
Sum = 53 57

Code-compliant detailing

There may be no crossties within joints



Test data classified by geometry and failure modes



failure mode	Int.	Ext.
B	12	19
BJ	20	19
J	10	13
BJ_a	11	6
Sum =	**53**	**57**

Beam-column connections

- with high-strength materials
- without trans. beams/slabs
- without eccentricity

110 specimens in total

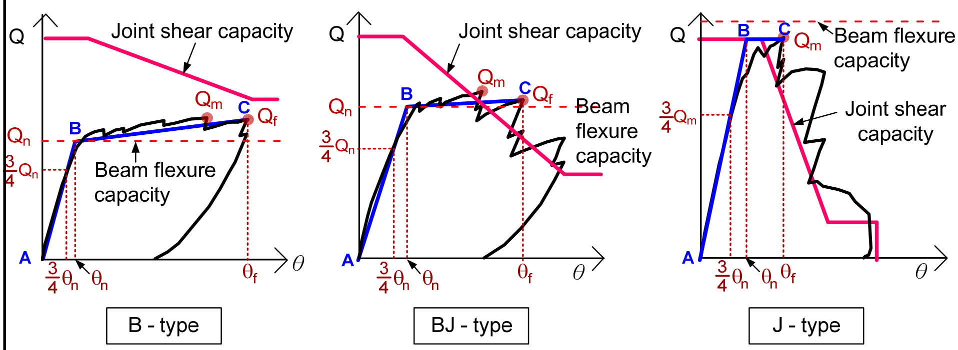
B-failure refers to beam flexure failure in the beam plastic hinges (joint remains elastic)

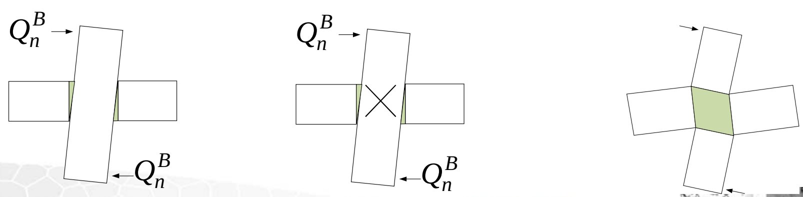
BJ-failure refers to joint shear failure along with yielding of beam reinforcement

J-failure refers to joint shear failure without yielding of beam reinforcement

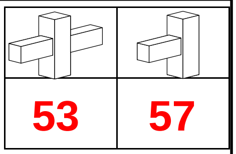
BJa = Bond or anchorage failure along with yielding of beam reinforcement

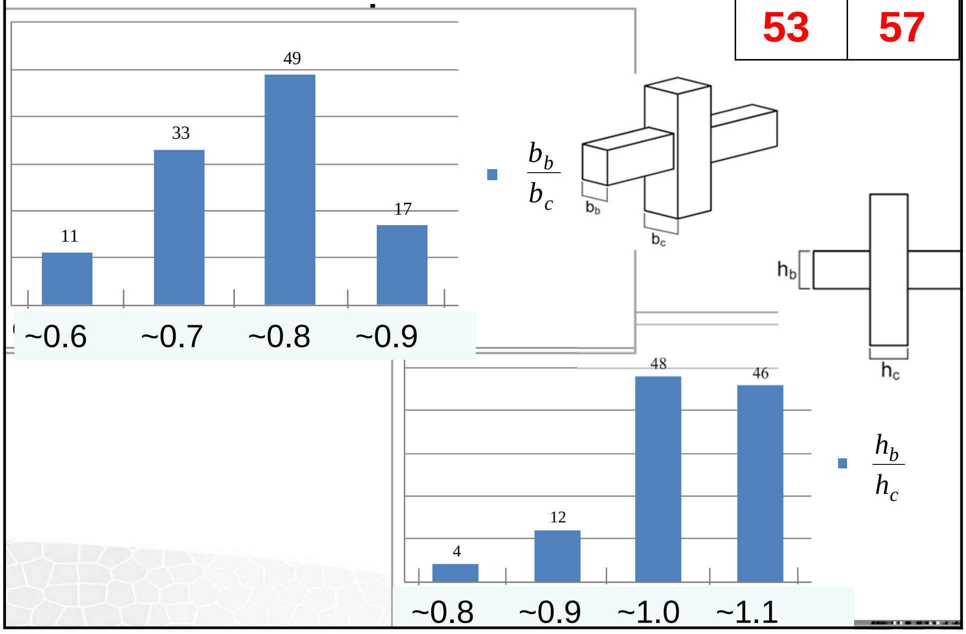
Failure modes, yield drifts and ductility





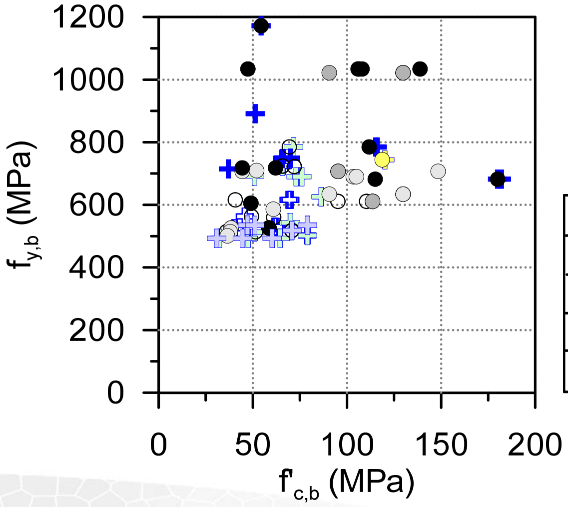
Ranges of width and aspect





Range of bar yield strengths and f'c

Beam bar f_y versus f'_c

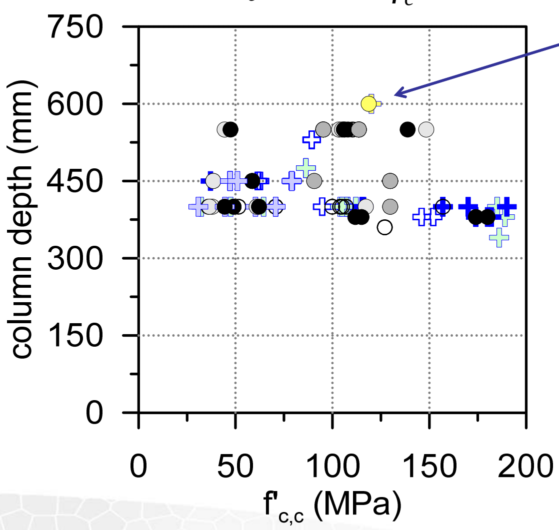


110 specimens in the database

Type	Int.(53)	Ext.(57)
B	✚ (open)	○
BJ	✚ (light green)	○ (light grey)
J	✚	●
BJ_a	✚ (light blue)	● (grey)

Range of column depth and f'c

Column depth h_c versus f'_c

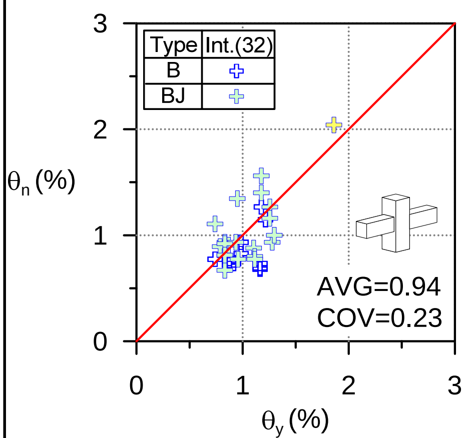


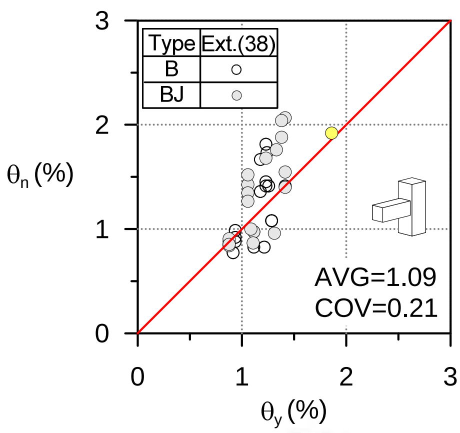
Our test specimens tested at NCREE in 2010

Type	Int.(53)	Ext.(57)
B	✚	○
BJ	✚	○
J	✚	●
BJ_a	✚	●

Story yield drift ratio

$$\theta_y = \varepsilon_y \cdot \frac{0.5 l_b}{h_b}$$

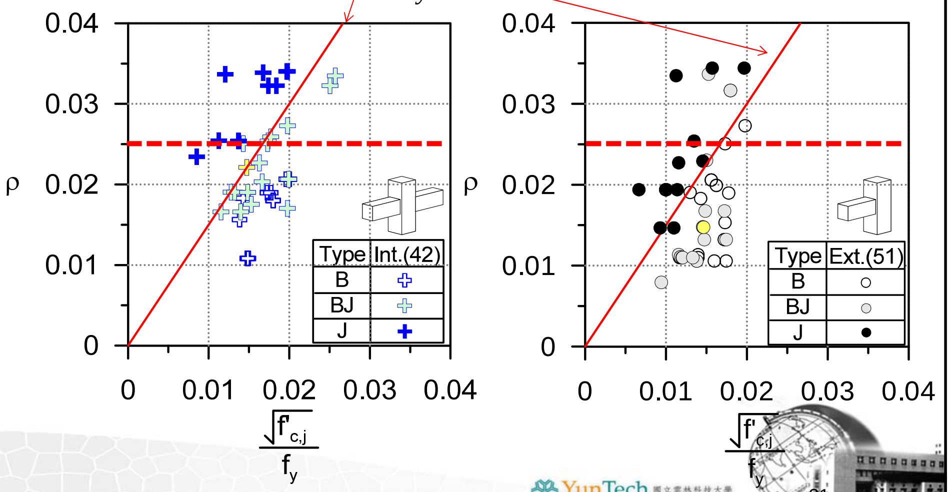




Priestley, M. J. N., "Brief Comments on Elastic Flexibility of Reinforced Concrete Frames and Significance to Seismic Design,"，Bulletin of the New Zealand National Society for Earthquake Engineering, V. 31,No. 4, Dec. 1998, pp. 246-259.

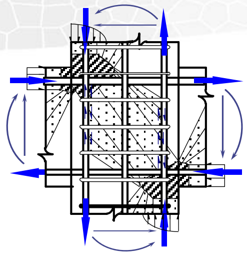
Maximum reinforcement ratio of beams

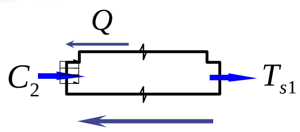
$$\rho_{max} = \text{the smaller of } \frac{1.5\sqrt{f'_c}}{f_y} \text{ and } 0.025 \quad (f'_c,\ f_y \text{ in MPa})$$



Outline

- ~~Why we need high-strength reinforcement~~
- ~~Database Investigation~~
- Prediction of joint strength and ductility
- Preliminary design recommendations



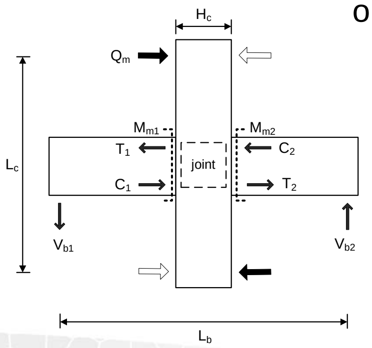


$$V_j = T_{s1} + C_2 - Q$$

Maximum shear force $V_{jh,m}$ acting on the joint

$$V_{jh,m} = T + C - Q_m$$

$$= Q_m\left[\frac{(l_b - h_c)}{z_b} \times \frac{l_c}{l_b} - 1\right]$$

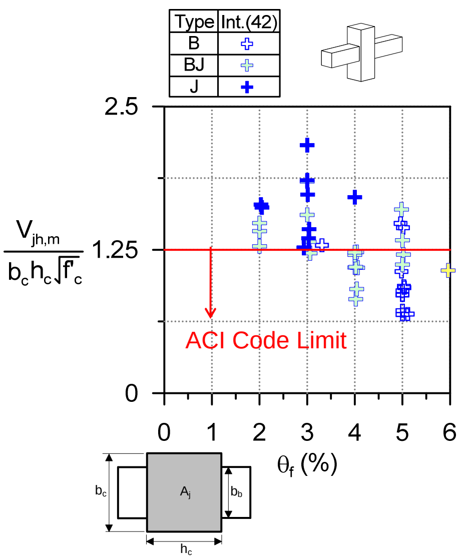


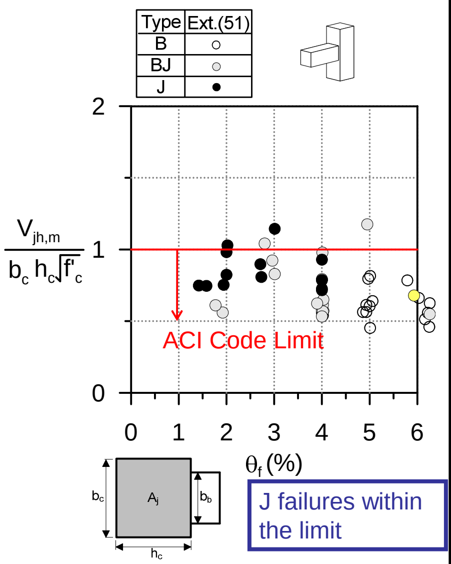
To get experimental joint shear stress

ACI 318 Code

$$b_j = b_c$$

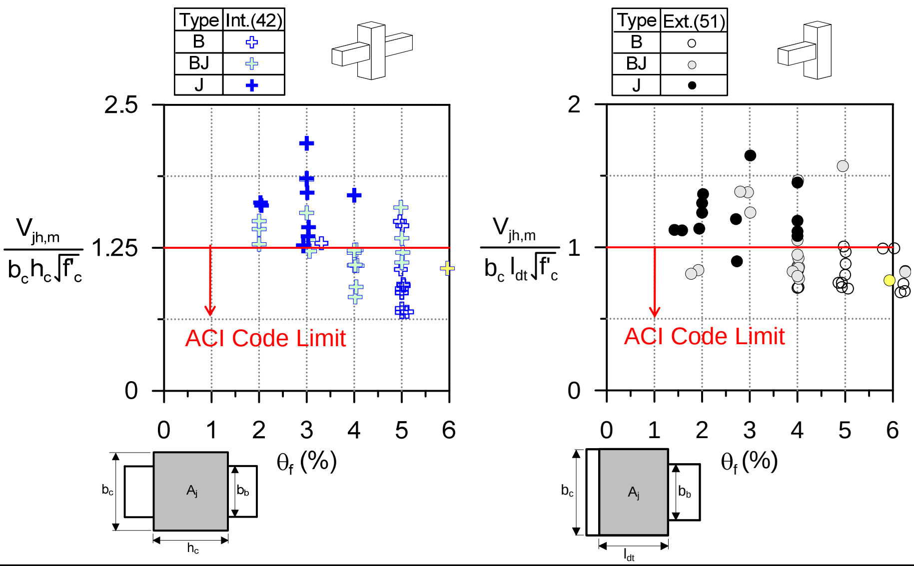
$$h_j = h_c$$





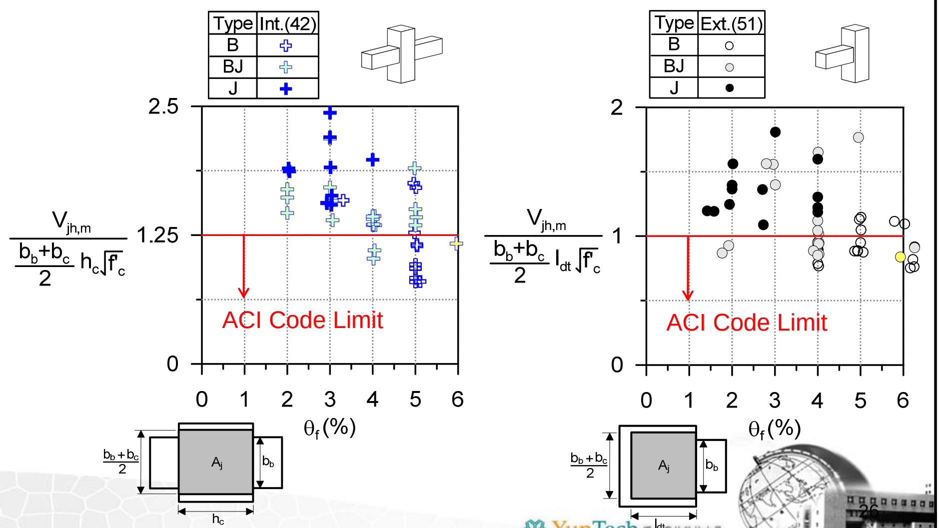
J failures within the limit

Effective joint depth for exterior joints

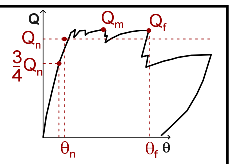


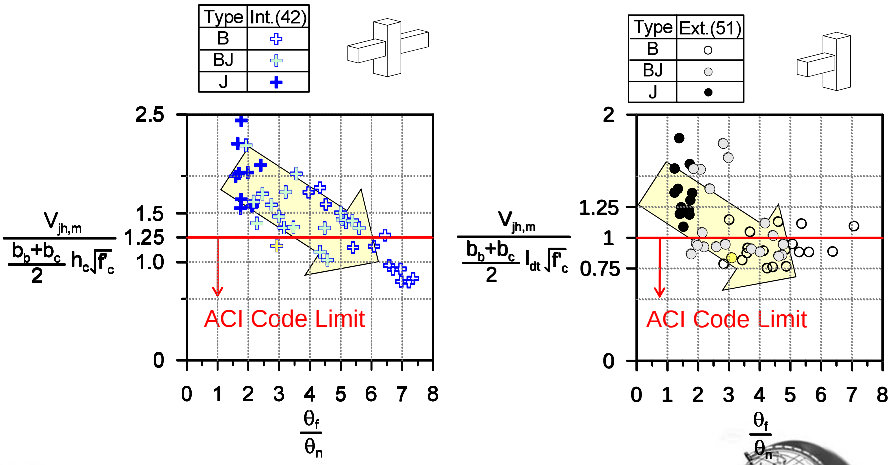
Effective joint depth for exterior joints

$$b_j = 0.5(b_b + b_c)$$

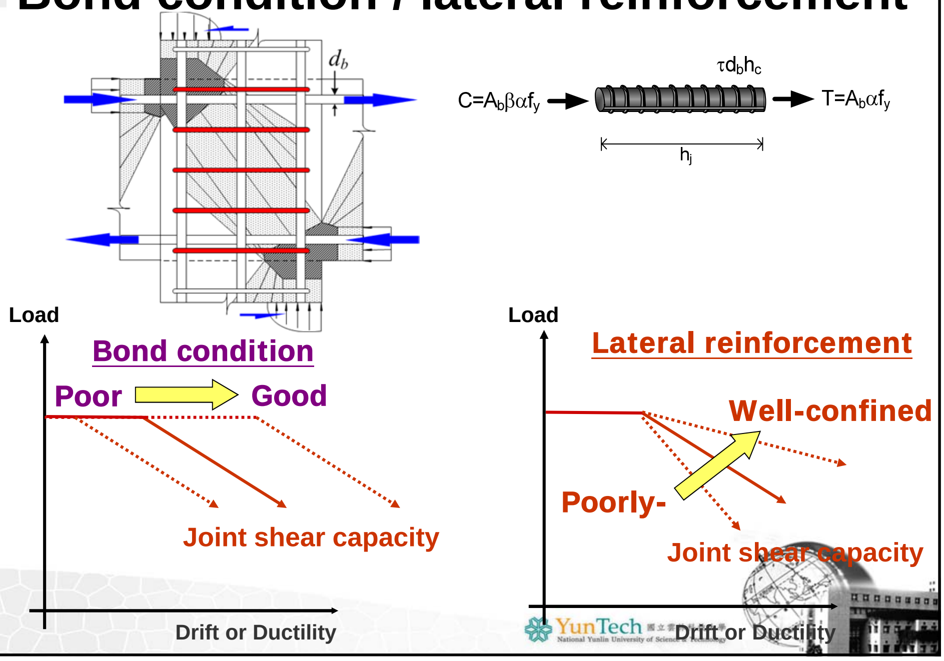


Joint shear strength v.s. ductility





Bond condition / lateral reinforcement

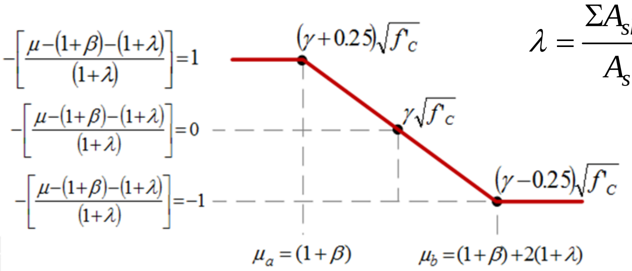


Degradation model for joint shear strength

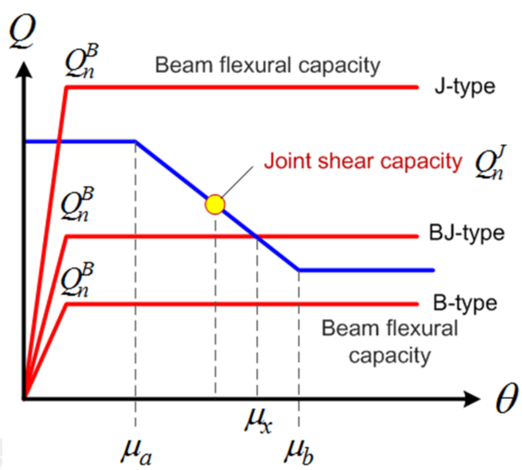
$$V_j = \left\{ \gamma - \left[\frac{\mu - (1+\beta) - (1+\lambda)}{(1+\lambda)} \right] \times 0.25 \right\} \sqrt{f_c'} A_j$$

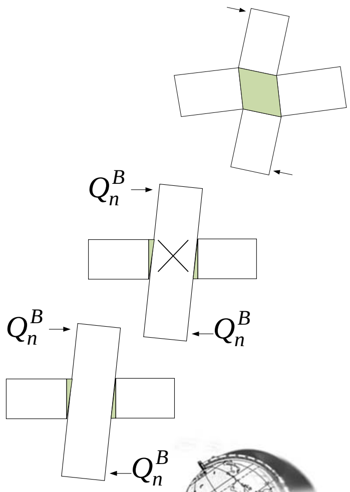
$$\beta = \frac{4}{\left(1 + A_{s,bot} / A_{s,top}\right)} \frac{\sqrt{f'_c}}{f_y} \frac{h_j}{d_b}$$

$$\lambda = \frac{\Sigma A_{sh} \bar{f}_{yt}}{A_s f_y}$$

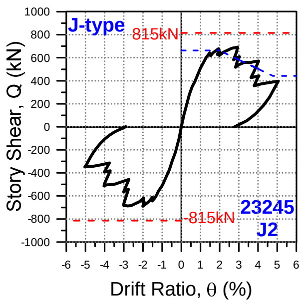


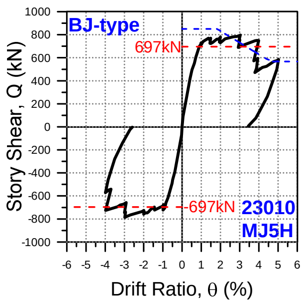
Failure mode predictions

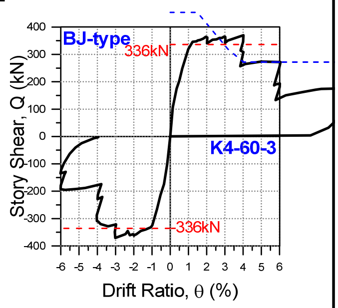


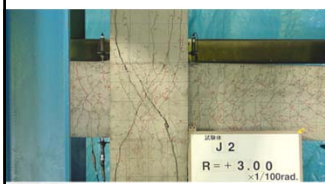


Selected examples

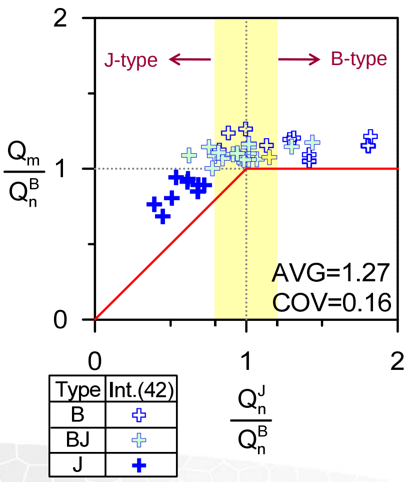


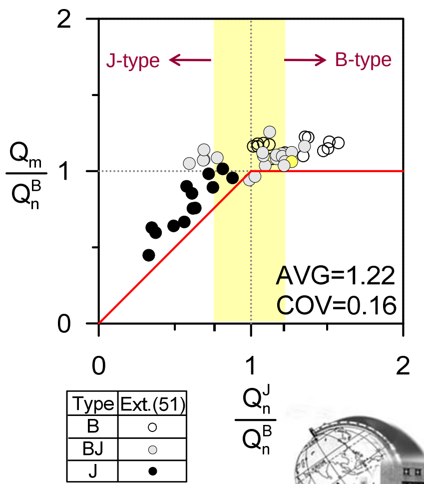




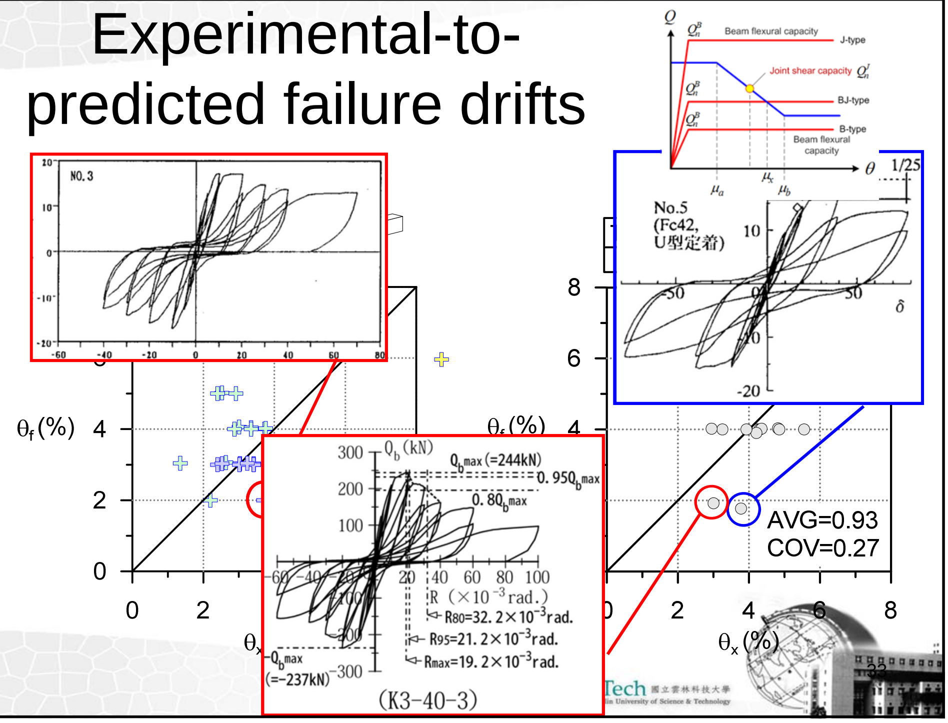


Experimental-to-predicted story shear strengths and failure modes





Experimental-to-predicted failure drifts



Outline

- ~~Why we need high-strength reinforcement~~
- ~~Database Investigation~~
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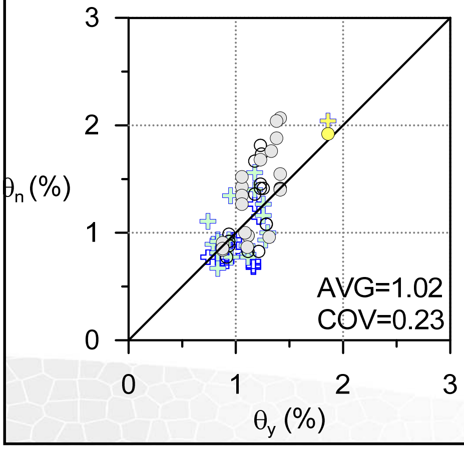
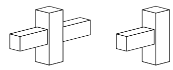
- Preliminary design recommendations

Preliminary recommendation 1

- Story yield drift ratio can be estimated



$$\theta_y = 0.5\varepsilon_y \frac{l_b}{h_b} \quad \text{(Priestley, M. J. N. 1998)}$$

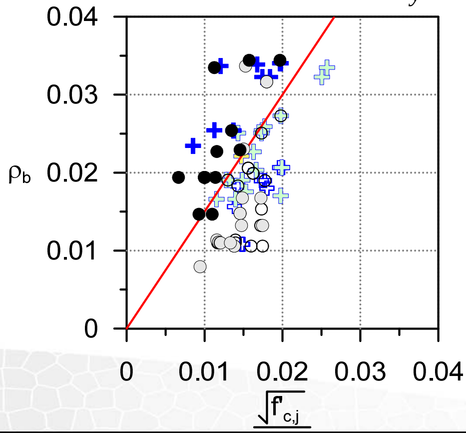


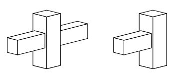
Type	Int.(32)	Ext.(38)
B	✚	○
BJ	✚	○

Preliminary recommendation 2

The maximum reinforcement ratio in a beam

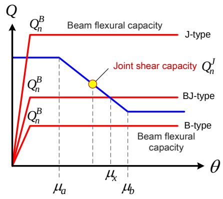
$$\rho_{\max} = \text{the smaller of } \frac{1.5\sqrt{f'_c}}{f_y} \text{ and } 0.025 \quad (f'_c,\ f_y \text{ in MPa})$$

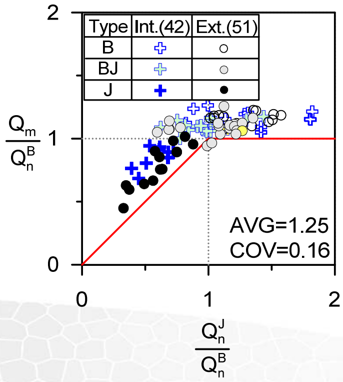


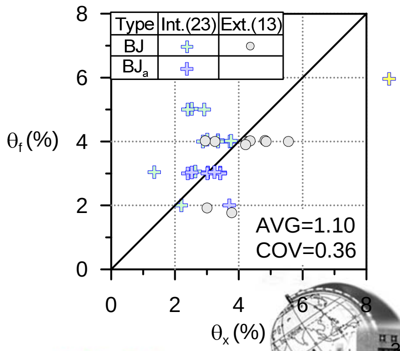


Type	Int.(42)	Ext.(51)
B	✚	○
BJ	✚	○
J	✚	●

Preliminary recommendation 3







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- Papers available from on-line database

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Thank you for your attentions

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