國立臺灣大學工學院土木工程學系

碩士論文

Department of Civil Engineering

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Master Thesis

越南 RC 構件耐震設計與鋼筋配置之案例研究
—以 EN 1998-1:2004 與 ACI 318-08 規範為標準

Seismic Design and Detailing of Reinforced Concrete Members in Vietnam Based on EN 1998-1:2004 and ACI 318-08 Codes

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ABSTRACT

Codes of practice of Europe and United States of America are shared with many countries in the world. In fields of civil and structural engineering, EN Eurocodes and ACI codes (American Concrete Institute) are commonly used and they have been constantly updated according to technology advancement of human beings. Many countries have adopted EN Eurocodes or ACI codes as their national codes.

The author would like to focus this study on the common construction problems in high rise buildings encountered in Vietnam, which deals with wide beam-column joints, beam-core wall joints, coupling beams and deep beams. These construction problems are first briefly described. The related seismic design and detailing are then compared and evaluated by using the EN 1998-1:2004 and ACI 318-08 codes. This study is expected to clarify some common mistakes and to improve the construction practice in Vietnam.

Keywords: wide beam-column joint, beam-core wall joint, anchorage, coupling beams, deep beams, transfer beams, strut-and-tie model, earthquake-resistant structures, high rise buildings.

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1.1.1 Brief on ACI

The American Concrete Institute (ACI) is the major agency for all concrete construction in the United States of America. It was established in 1904 to serve and represent user interests in the field of concrete. According to ACI-A Century of Progress [1], the history of an organization is a combination of events linking people, ideas, and activities. The ACI, and its predecessor, the National Association of Cement Users, were born of ideas-the concept that better concrete for more durable, maintenance-free structures is possible. The ACI reflects developments and knowledge within the concrete industry and the field of engineering, while at the same time influences those developments.

The ACI publishes many different standards, but the most commonly referenced standard used by architects and engineers is the ACI 318 [2] “Building Code Requirements for Structural Concrete.” It is updated continuously and the latest version is ACI 318-08 updated in 2008. Almost all building codes, including the IBC (International Building Code), refer to ACI 318 code as the basis for structural design of concrete members.

1.1.2 Brief on ACI 318-08

ACI 318-08 code includes: Introduction, 22 chapters and 5 appendixes:

Introduction

Chapter 1-General requirements

Chapter 2-Notation and definitions

Chapter 3-Materials

Chapter 4-Durability requirements

Chapter 5-Concrete quality, mixing, and placing

Chapter 6-Formwork, embedments, and construction joints

Chapter 7-Details of reinforcement

Chapter 8-Analysis and design-General considerations

Chapter 9-Strength and serviceability requirements

Chapter 10-Flexure and axial loads

Chapter 11-Shear and torsion

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1.2 Brief on EN and EN 1998-1:2004

1.2.1 Brief on EN

The EN Eurocodes describes the design method for buildings and civil engineering work. They consist of 10 different groups [3], which are:

EN 1990-Eurocode 0-Basis of structural design

EN 1991-Eurocode 1-Actions on structures

EN 1992-Eurocode 2-Design of concrete structures

EN 1993-Eurocode 3-Design of steel structures

EN 1994-Eurocode 4-Design of composite steel and concrete structures

EN 1995-Eurocode 5-Design of timber structures

EN 1996-Eurocode 6-Design of masonry structures

EN 1997-Eurocode 7-Geotechnical design

EN 1998-Eurocode 8-Design of structures for earthquake resistance

EN 1999-Eurocode 9-Design of aluminum structures

Each Eurocode consists of several parts. There are 58 parts in all. The work with the Eurocodes started in 1975. The first publications came in the mid 80's. By 2006 the EN Eurocodes Parts are expected to be published. By 2010 the Eurocodes are expected to be fully implemented and will replace all national codes. Some of the aims and benefits of the Eurocodes are to:

- Provide common design criteria and methods.

- Provide a common understanding of construction products.

- Facilitate the exchange of construction services.

- Be a common basis for research and development.

- Allow the preparation of common design aids and software.

- Increase the competitiveness of the European civil engineering firms, contractors, designers and product manufacturers in their world-wide activities.

Due to difficulties in harmonizing the calculation methods and level of safety, National Determined Parameters (NDP) has been included in the Eurocodes. The NDP's can be found in a National Annex, which is a national standard and has to be applied in conjunction with the European standard.

The European Commission has supported, from the beginning, the elaboration of Eurocodes, and contributed to the funding of their drafting. It continues to support the task mandated to CEN to achieve the publication of EN Eurocodes. It will watch the implementation and use of the EN Eurocodes in the Member States. Table 1.1 will shows history overview of the creation of the Eurocodes.

1.2.2 Brief on EN 1998

EN 1998-Eurocode 8-Design of Structures for Earthquake Resistance includes 6 parts as following [3]:

EN 1998-1: Eurocode 8-Design of Structures for Earthquake Resistance-Part 1: General rules, seismic actions and rules for buildings. Eurocode 8 applies to the design of buildings and civil engineering works in seismic regions. Its purpose is to ensure that in the event of earthquakes: Human lives are protected; damage is limited; and structures important for civil protection remain operational. EN 1998-1:2004 is EN 1998-1 Eurocode 8-Structures for Earthquake Resistance-Part 1: General rules, seismic actions and rules for buildings, its version is year of 2004.

EN 1998-2: Eurocode 8-Design of Structures for Earthquake Resistance-Part 2: Bridges within the framework of the general requirements set forth in Part 1.1. This part of the code contains design principles, criteria and application rules applicable to the earthquake resistant design of bridges.

EN 1998-3: Eurocode 8-Design of Structures for Earthquake Resistance-Part 3: Assessment and retrofitting of buildings. This document provides criteria for the evaluation of the seismic performance of existing individual building structures, and describes the approach in selecting necessary corrective measures.

EN 1998-4: Eurocode 8-Design of Structures for Earthquake Resistance-Part 4: Silos, tanks and pipelines. This standard includes the additional criteria and rules required for the seismic design of this structure without restrictions on their size, structural types and other functional characteristics. For some types of tanks and silos, however, it also provides detailed methods of assessment and verification rules.

EN 1998-5: Eurocode 8-Design of Structures for Earthquake Resistance-Part 5: Foundations, retaining structures and geotechnical aspects. This Part of Eurocode 8 establishes the requirements, criteria, and rules for siting and foundation soil of structures for earthquake resistance. It covers the design of different foundation systems, earth retaining structures and soil-structure interaction under seismic actions.

EN 1998-6: Eurocode 8-Design of Structures for Earthquake Resistance-Part 6: Towers, masts and chimneys. This document deals with material related Eurocode parts dealing with towers, masts and chimneys. Design rules for the earthquake resistant design of tall, slender structures: Towers, including bell-towers, masts, industrial chimneys and lighthouses constructed in reinforced concrete or steel.

1.2.3 Brief on EN 1998-1:2004

EN 1998-1:2004 [4] is Eurocode 8-Design of Structures for Earthquake Resistance-Part 1: General rules, seismic actions and rules for buildings, and the latest version is EN 1998-1:2004 updated in 2004.

EN 1998-1:2004 includes 10 parts and 3 annexes:

1. General.
2. Performance requirements and compliance criteria
3. Ground conditions and seismic action
4. Design of buildings
5. Specific rules for concrete buildings
6. Specific rules for steel buildings
7. Specific rules for composite steel – concrete buildings
8. Specific rules for timber buildings
9. Specific rules for masonry buildings
10. Base isolation

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Annex B (Informative) Determination of the target displacement for nonlinear static (pushover) analysis.

Annex C (Normative) Design of the slab of steel-concrete composite beams at beam-column joints in moment resisting frames.

The EN 1998-1:2004-Structures for Earthquake Resistance-Part 1: General rules, seismic actions and rules for buildings, Section 5-Specific rules for concrete buildings, is focused to study in thesis.

1.3 Vietnam seismic risk

1.3.1 Earthquake situation in Vietnam

The report of Ministry of Construction of Vietnam [5]: According to Vietnam seismic zonation map as shown in Figure 1.1 in TCXDVN 375:2006-Part 1 [6], Vietnam has only a few areas of the Northern regions are predicted to be able to earthquake level VIII (MSK-64 scale). Shocks caused by earthquakes at some Northwest locations can reach level IX, also the majority of Vietnam's territory can occur, and the soil conditions are very weak. Thus, the earthquake happened in Vietnam is not strong intensity and the amount is not much compared to many parts of the world, usually ranging weak to moderate intensity. Frequency of earthquakes with occurred strong intensity is very low.

Results of the research and forecasting earthquakes in Vietnam, since 2005 show that year of 114 to 2003, with measurements or historical data, the earthquakes were recorded 1.645 with magnitude of over 3 Richter. The earthquake in Tuan Giao (Dien Bien province, Northwest of Vietnam) in 1983 with intensity of 6.8 Richter and the largest quake in Vietnam was recorded. In addition, there were also the earthquakes in 1935, 2001 with magnitude of 6.7-6.8 Richter ever happened in Dien Bien province.

The earthquakes have intensity from 4.6 to 4.8 Richter in other places: Bac Giang-1961, Son La-1983 and 2009, Dien Bien-2001 (Northern Vietnam), Nghe An-2005 (South of Northern Vietnam).

Each code has difference classification for seismic intensity, Russian Federation: MSK-64 scale, France: MM scale, United State of America (UBC-Uniform Building Code): Zones, Japan: JMA scale. Table 1.2 shows conversion between peak ground acceleration and earthquake intensity according to MSK-64 scale and MM scale.

1.3.2 Conclusions

Vietnam is located in low-to-moderate seismic regions if compared with some Asian countries like Japan, Taiwan, Indonesia, etc. Only some specific zones are located in strong seismic regions, like Dien Bien province in Northwest Vietnam.

In Vietnam, however, according to TCXDVN 375:2006-Part 1 [6], based on the design ground acceleration $a_g=\gamma_I a_{gR}$, design for earthquake-resistant structures is divided into three categories as following (Figure 1.1):

- $a_g \geq 0.08g$ (Strong seismicity): Shall be calculated and detailed for earthquake-resistant structures.

- $0.04g \leq a_g < 0.08g$ (Low seismicity): Reduced or simplified seismic design procedures for certain types or categories of structures may be used.

- $a_g < 0.04g$ (Very low seismicity): The provisions of TCXDVN 375:2006 need not be observed.

where a_{gR} is determined from ground acceleration classification map of Vietnam (Appendix H, part 1, TCVN 375:2006-Part 1) or Table of ground acceleration classification according to administrative sites (Appendix I, part 1, TCXDVN 375:2006-Part 1).

1.4 Situations and difficulties in Vietnam's civil engineering

1.4.1 Normative laws and regulations on technical standards for the earthquake prevention and resistance for structures

1.4.1.1 State's legal documents:

Table 1.3 indicates the released time for legal documents in Vietnam's civil and industry field. Before 1996, almost codes are used from former Soviet Union. From 1996 to 2004, only standards and codes of 7 countries and international organizations have been adopted in Vietnam, such as: England, United State of America, Australia, Japan, ISO, etc. From 2005 to present, standards and codes from any countries, international organizations, regional codes organizations have been allowed in Vietnam according to requirements of Ministry of Construction (Vietnam).

1.4.1.2 Design standards of reinforced concrete structures for earthquake resistance

Design standards related to reinforced concrete structures and reinforced concrete structures in high rise buildings are

- TCVN 2737:1995 Load and action-Design standard (based on former Soviet Union).

- TCVN 5574:1991 Reinforced concrete structures-Design standard (former Soviet Union.

- TCXD 198:1997 [7] High rise building-Guide for design of monolithic reinforced concrete structures (former Soviet Union).

The provisions of calculation and design for reinforced concrete structures are used standards of former Soviet Union, in fact, main contents related earthquake resistance in TCXD 198:1997 is based on СНиП II-7-81* (Standards and Regulations for Construction, Chapter 7, Part II) of former Soviet Union.

Until years of 2005, 2006, Vietnam has issued three standards for the design of reinforced concrete structures is based on the standards of Russian Federation and Eurocodes:

- TCXDVN 356:2005 [8] Concrete and reinforced concrete structures-Design standard (Russian Federation).

- TCXDVN 375:2006 [6] Design of structures for earthquake resistance-Part 1: General rules, seismic actions and rules for buildings (Eurocodes).

- TCXDVN 375:2006 Design of structures for earthquake resistance-Part 2: Foundations, retaining structures and geotechnical aspects (Eurocodes).

TCXDVN 375:2006-Part 1 guided specific cases to consider the impact of earthquakes and measures earthquake resistant design for buildings. Accordingly, structures in low seismicity regions, when the ground acceleration based soil type A does not exceed 0.78 m/s^2, it can use the design is subjected to mitigation earthquake or simplified for some categories, types of structures. For structures in the earthquake zones are very low, when the ground acceleration based soil type A does not exceed 0.39 m/s^2, need not comply with the terms of this standard.

In which, Part 1 and Part 2 of TCXDVN 375:2006 were translated on the basic of EN 1998-1 and EN 1998-5 respectively, while TCXDVN 356:2005 [8] were translated on the basic of СНиП 2.03.01-84* of Russian Federation. Because of large difference gaps between TCXDVN 356:2005 and TCXDVN 375:2006, many local engineers do not know how to design comply with standards.

1.4.2 Observance of legal documents, technique standards for the prevention and resistance of earthquake

In Letter issued 2008 [5] by the Ministry of Construction of Vietnam, the situation of observance of legal documents, technique standards of earthquake prevention for construction projects in Vietnam in recent years has been not good.

At the stage construction period 1954-1976, with the structural solution of the reinforced concrete large panels were assembled forming the apartment zones with 1 through 5 stories. However, most of the projects are low building and not designed earthquake resistance.

At the period 1976-1986, in Northern Vietnam, almost houses with reinforced concrete large panels assembled. Some of them were calculated for earthquake resistance. Vietnam's first building designed subject to earthquake load was 11 floors in Giang Vo-Hanoi capital (now is Hanoi Hotel). Most of the projects designed for earthquake resistance in the north of Vietnam. In the southern Vietnam, almost buildings were built previously not interested in earthquake resistance.

In the phase from 1986 to 1997, a number of foreign investment projects deployed in Vietnam. The high-rise buildings were designed for earthquake resistance.

In the construction phase from 1997 to present, the construction work was developed on the number, category and level of works. Vietnam has appeared more and more high rise buildings over 20 floors. Particularly in Hanoi, many high rise buildings using core wall slip solutions combined with assembled floors and columns. The projects are constructed according to this solution has disadvantages are difficult to control the quality for joints, so it to be limited in use. The projects under construction in Hanoi and elsewhere during this period are most designed for earthquake resistance at level VII (MSK-64 scale). The standards for earthquake resistance are applied mostly standards of the former Soviet Union and UBC (Uniform Building Code) of the United States of America.

Pursuant to the report of the Committee of Provinces and Cities directly under the Central Government (42/63 local reports), the construction works at the local before TCXDVN 375:2006 becomes effect, are not interested in earthquake resistant design, except in large cities like Hanoi (northern), Da Nang city (central), Ho Chi Minh city (southern), are done well.

1.4.3 Conclusions

(1) Generally, construction's law and standards are not completed. Design standards for concrete and reinforced concrete structures is not consistent, currently still at the stage to continue shifting standards Eurocodes. It will take several years for Vietnam to have a full set of standards for design and construction of reinforced concrete structures.

(2) In the present, because of large difference gap between TCXDVN 356:2005 and TCXDVN 375:2006, many local engineers do not know how to design according to standards.

(3) In the past and also at the present, beside TCXDVN 375:2006-Part 1 (EN 1998-1) comes into effect, the design earthquake-resistant structures for the projects in Vietnam has mainly based on foreign standards such as СНиП II-7-81* of the former Soviet Union, the United States of America such as various versions of UBC-1985, UBC-1988, UBC-1991 and UBC-1997 [9]. By СНиП II-7-81* in accordance with the design standard system of Vietnam's current so designers often use more than other standards. The contents of the СНиП II-7-81* were also included in the design of earthquake resistance in TCXD 198:1997. The EN 1998-1 (TCXDVN 375:2006 Part 1) has been effected in 2006, due to inconsistency so that it was not used commonly. ACI 318 code has only been applied in Vietnam for some projects.

(4) Obviously, Vietnam has little experience in design for earthquake resistance. Due to limited capacity should still exist a number of traffic works, irrigation, civil engineering... designed by local consultancy organization is not considered when designing earthquake resistance. Moreover, documents related new standards issued in 2005, 2006 has not been released, such as evaluation and strengthening the structures, and guiding detail for earthquake resistance.

(5) In the set of EN Eurocodes, Vietnam just compiled and put to use some parts in EN 1998. The rest of the Eurocodes are in its compilation. Particularly, EN 1992-1-1 [10] Eurocode 2-Design of Concrete Structures-Part 1-1: General Rule and Rules for Buildings, still does not adopt in Vietnam legally, it has unofficial document in Vietnamese [11], while EN 1998-1 has been adopted in Vietnam as name TCXDVN 375:2006-Part 1 [6]. At present, two guide books was released, first book [12] is used for TCXDVN 375:2006, and other one [13] is used for TCXDVN 356:2005. Therefore, it can say that engineers are currently had to use many other standards in earthquake-resistant design for the projects.

(6) For each standard, in addition to issuing the new standards will also need to change the entire textbook in universities and of course the whole curriculum, hold on research and training courses on contents of codes. This so far has not done so that it already makes many difficulties for Vietnamese engineers in designing, particularly earthquake-resistant design for structures.

(7) Since all of these reasons, EN 1998-1 is official in Vietnam but not implemented yet thorough and effective; ACI 318 code for earthquake-resistant structures is used rarely in Vietnam. There are some books about ACI 318 codes was released [14], [15]. If ACI code is applied then often using options are available in the software of analysis and design of structures such as SAP2000, ETABS, etc. These are limited of the author in this thesis.

(8) In addition, it can say that, like other developing countries, in civil engineering, economic and time factors would normally considered more than these technical factor, which as a general rule of development. This is a very difficult problem for engineers to execute the work under the provisions of the standard

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CHAPTER II
TYPICAL CONSTRUCTION PROBLEMS FACED IN VIETNAM

2.1 Problem 1: Wide beam-column joints, beam-core wall joints

Beam-column joints and beam-core wall joints are an important part of a reinforced concrete moment resisting frame subjected to earthquake loading. Design and detailing provisions on beam-column joints and beam-core wall joints in wide beam cases in codes do not adequately address prevention of anchorage failure and shear failure in regions during other level earthquake shaking. In Vietnam, many high rise buildings used as offices, hotels, residential and commercial buildings are often designed with a beam whose width is larger than beam height, this beam is called wide beam with wide beam width is wider than column size. As required by the architecture, or in other words, economic factors, height of story is usually 3.3 m, rarely designed 3.6 meter or 3.9 meter in height. When it is needed big space to facilitate the function layout of project, the span of beam should pass greater than normal, such as column grid is greater than 8m. To handle this problem, there are many ways, such as pre-stressed beams, flat floor, etc. However, engineers often choose the safest way is reinforced concrete wide beams. For example, the height of the storey of 3.3 meter, the average height of beam is only about 0.4 through 0.5 meter to ensure clear spans and function of the building. For this beam height, in order to the beam can be passed a large span, for instance 8 meter, of course, it will be needed to expand the beam width to 1 meter or reach 1.5 meter. While the column size can not expand the area to equal with beam width, in many cases, only about 0.5 meter to 0.8 meter of column size is enough for strength resistant capacity of column. On contrary, in many cases, column size are expanded to reach wide beam width due to structural designers do not know how to solve problems that the wide beam width is wider than column size.

Figure 2.1 shows an example of a reinforced concrete frame with the wide beam-column joints, and in connection types, the wide beam width is wider than column section, part of beam longitudinal reinforcements passes outside of the column core, as shown in Figure 2.2 (Gentry and Wight [16]). Comparing to the normal conditions is beam width is smaller columns section (Figure 2.3), in other word, all of beam section is located inside the column area, this problem has been indicated in the standards and codes. In the cases of the wide beam width is wider than the column size (Figure 2.4), the issues should be resolved is detailing of reinforced steels in wide beam which are passed outside of column core for joints of exterior and corner (Figure 2.4.b, c, e, f), especially for earthquake-resistant structures.

In the cases of the wide beam, some standard's provisions are considered the effective wide beam width and the deviation between the center of columns and wide beams, but also needed to consider the issue of wide beam's reinforced steel anchorage. How to calculate and detail for reinforced bars that are outside of column area to ensure seismic resistance (Figure 2.5, 2.6), including top and bottom layers, in the load case of earthquake, the beam moment will be changed sign.

Beam-core wall joints will be designed normally as beam connects at intersection point between two walls, or wall section can be expanded as column to fit with beam (Figure 2.7.a). However, it is not always able to do so. In fact, there are many cases that beam connects to core wall at other positions as shown in Figure 2.7.b, 2.7.c, even above coupling beam. In such cases, the best way that let consider them as regular cases in Figure 2.7.a, it means that no any problems occur or there is nothing difference between connections in Figure 2.7.b and Figure 2.7.c.

Some engineers also solve the connection in Figure 2.7.b, 2.7.c by detailing of beam’s reinforcing bars in order to the joint between core wall and beam becomes hinge (Figure 2.8.c). The “hinge connection” in Figure 2.8.c will lead reinforcement ratio less than so much “rigid connection” in Figure 2.8.b in beam D1F3. Briefly, how to solve the joints in Figure 2.7.b, 2.7.c, and put the hinge at joint (Figure 2.8.c) is suitable or not, especially for design of earthquake-resistant structures.

Table 2.1 illustrates status of problem 1 in 3 codes of Vietnam, Europe and United States of America.

Lastly, contents will be solved in this study:

(1) For the wide beam-column joints:

- Anchorage issues of wide beam’s reinforced bars passed outside of the column core in 4 typical joints: Exterior joint (Figure 2.4.b), corner joint (Figure 2.4.c), roof-exterior joint (Figure 2.4.e) and roof-corner joint (Figure 2.4.f).

- Related issues on design of wide beam-column joints.

(2) For the beam-core wall joints:

- Detailing issues of beam-core wall joints: Rigid or hinge connections.

- Related issues on design of beam-core wall joints: core wall size at connection (Figure 2.7.b), eccentric joints (Figure 2.7.c).

(3) The wide beam-column joints could be designed in seismic regions or not.

2.2 Problem 2: Coupling beams

According to Fortney [17], in high rise buildings, core wall and coupling beams are indispensable parts in the structural system resist lateral forces (Figure 2.9). Coupling beams if properly reinforced to provide them with sufficient strength and stiffness, can increase the lateral stiffness of the building significantly. Lateral deflection of walls induces large moments and shears in the coupling beams as they resist imposed deformations. Figure 2.10 illustrates the different mechanisms for resisting overturning moments for uncoupled wall piers and the coupled system. Coupled and uncoupled systems act similarly in terms of resisting gravity loads; as illustrated in Figure 2.10, the difference between the two structures is realized only when resistance of lateral loads is considered. When the coupling beams over the height of the core wall system are proportioned appropriately to attain the desired behavior of the coupled core wall system, the coupling beams will form plastic hinges, in shear, simultaneously while going through very similar beam end rotations. This behavior results in a desirable distribution of energy dissipation (in the coupling beams) over the height of the building as opposed to the energy dissipation being concentrated at the base of the flexural wall piers. In order to achieve the desired behavior of the coupled core wall system, the coupling beams must be designed and proportioned for adequate stiffness and strength. However, the coupling beams must also yield prior to the wall piers and demonstrate stable hysteretic response and good energy-absorbing characteristics.

With actual conditions in Vietnam and as well as in other countries, ensuring quality of reinforced concrete core wall and the coupling beams, especially for positions linked between elements, such as core wall and beam, is not easy. In most cases, the ratio of reinforcement in the coupling beams and at joint of core wall with beam is so high.

Because of these difficulties, contractors and other responsibility engineers were free to change reinforcement ratio and position of reinforcing bars to more easily construct. It is so clearly wrong, but also that designer has product is not perfect. Figure 2.12, 2.13 and Figure 2.14 show the differences between codes (Figure 2.11.a and

2.11.b), drawing and actual site. There are also cases that engineers have to bend anchored development length for diagonal bars in the coupling beams by they are too long, difficult installation, steel congestion in wall, etc. (Figure 2.15). In fact, the issue of anchoring for diagonal reinforcements in the coupling beams also faces problem such as the development length exceeds concrete section of core wall. In this case, bend of diagonal bars with appropriate shapes are forced to anchor them into concrete section to ensure adequate anchored length of diagonal bars according to requirement of the standards (Figure 2.15). In addition, sometimes, the coupling beams are replaced by slabs in practice, due to requirements from architect or structural designer, or by contractor for easy to build. In order to ensure quality during construction at site, what matters to consider for the coupling beams are such as: The necessary of diagonal reinforcements, bending angle of diagonal reinforcements, and the coupling beams can be removed in some cases or not… Generally, the coupling beams have not been considered carefully in both stages of design and construction for high rise buildings in seismic regions. Many coupling beams had been became lintel beam in core wall in lateral loading resisting system by inadequacy understanding, a maijor reason is by Vietnamese standard. Table 2.1 also illustrates status of problem 2-coupling beams in 3 codes of Vietnam, Europe and United States of America.

Briefly, problems about the couppling beams are

(1) Necessitating the use of diagonal bars in the coupling beams, detailing diagonal bars and changing content of diagonal bars in the coupling beams.

(2) Discontinuity (cut off) diagonal bars at mid-span.

(3) Anchored bend of diagonal bars into core wall.

(4) Anchorage of horizontal bars.

(5) Drop of the coupling beams and replace by slab.

2.3 Problem 3: Deep beams in high rise buildings

Transfer structures are commonly used in the world, especially for regions of non-seismicity and low-to-moderate seismicity, such as Southeast Asia: Bangkok, Malaysia, Singapore, especially Hong Kong, and some regions in mainland China. This structure can be used for low rise and high rise buildings. Transfer structures can be designed as either shear or flexural members. Some types of transfer structures are deep beams, transfer girders, transfer plates, transfer beams, transfer boxes and transfer trusses.

In Vietnam, the concept of deep beams is usually only in materials of foreign country or textbooks that translated from other languages, Vietnamese engineers rarely use it in projects. Overseas firms have been often designed transfer structures as transfer beams, deep beams in high rise buildings. Categories of the deep beams are one span, continuous span as transfer girders, transfer beams (Figure 2.16, 2.17.b, 2.18 and 2.19). In fact, projects using the transfer structures are not much in Vietnam. Some reasons to limit in using these structural types that are complexities of its design, ensuring quality in sites, and less experience. In Vietnam, projects using transfer structures often designed by foreign consultant company, or at least concept or preliminary design by foreign engineer, had problem due to congestion of steel in beam, resulting in concrete quality was to poor and it was solved again by using self-compacting concrete. Some transfer structures as deep beams, transfer beams and transfer plates are used in Vietnam such as Trung hoa-Nhan chinh, Golden Westlake (Hanoi), Mannor 2, The Everich, Hung Vuong plaza, Saigon Pearl, Kenton residences, Sailing Tower (Ho Chi Minh city), etc.

The calculations for transfer beams as flexural member (as conventional beam) or single deep beams are simple but rarely appearances in high rise buildings.

Otherwise, transfer beams as multiple span deep beams appear much more. When it has a standard form like theory or sample in some researches then problems can be solved by different ways of calculation, Figure 2.18 presented model of strut-and-tie method. Nevertheless, the fact is that design of transfer beams is not easy as theory, Figure 2.19, 2.20 and Figure 2.21 present one example of transfer beams had been done in Vietnam, basic design by foreigner designer, and detail design by local consultant. The questions in complex cases are how to simplify, calculate, and detail not only for deep beams, but also for transfer structures, especially reinforcement ratio is much in the deep beams.

In strong seismicity regions, Vietnam is an example now, design of earthquake-resistant structure in high rise buildings, maybe need to consider to provisions for using the deep beams, transfer structures. It should be limited to design them in moderate to strong earthquake zones. Moreover, designer should be also selected simple and friend structural models to computer models in order to ensuring the safety factor for quite complex structural system. Table 2.1 illustrates status of problem 3-deep beams in 3 codes of Vietnam, Europe and United States of America.

In brief, issues will be solved in this thesis:

(1) Analytic model for complicated deep beams.

(2) Application of the transfer structures in seismic regions.

2.4 Seismic design category

The EN 1998-1:2004 provides the option to design reinforced concrete buildings for a combination of strength and ductility relationship by defining three alternatives ductility classes. Three dissipation classes are (Spathelf [18], Elghazouli [19]):

- Low (ductility class low (DCL)) in which virtually hysteretic ductility is intended and the resistance to earthquake loading is achieved through the strength of the structure rather than its ductility.

- Medium (DCM) in which quite high levels of plasticity are permitted and corresponding design and detailing requirements are imposed.

- High (DCH) where very large inelastic excursion are permitted accompanied by even ore onerous and complex design and detailing requirements.

For reinforced concrete buildings designed for low energy dissipation capacity and low ductility (DCL), no specific seismic detailing requirements have to be met. Clause 5.2.1(2)P (EN 1998-1:2004) said that concrete buildings may alternatively be designed for low dissipation capacity and low ductility, by applying only the rules of EN 1992-1-1:2004 [10]. For buildings which are not base-isolated, design with this alternative, termed ductility class L (low), is recommended only in low seismicity cases. According to Clause 5.3.1, seismic design for DCL, following EN 1992-1-1:2004, without any additional requirement, except provision on use of reinforcing steel class in primary seismic elements, is recommended only for low seismicity cases. In contrast, structures designed for relatively high energy dissipation and overall ductile behavior are classified into two ductility classes, namely DCM and DCH, depending on the hysteretic dissipation capacity. Specific earthquake-resistant detailing provisions apply to both of these ductility classes, enabling the structure to dissipate hysteretic energy under repeated reversed loading without developing brittle failure modes. Both DCM and DCH are presented by the behavior factor q, and q depends on structural types, such as frame system, dual system (frame or wall equivalent), ductile wall system (coupled or uncoupled), system of large lightly reinforced walls, inverted pendulum system, torsion flexible system.

ACI 318-08 requires that all structures shall be assigned to a seismic design category (SDC), including 6 classes of SDC: A, B, C, D, E, F. SDC A, B corresponds to the lowest seismic hazard, SDC C may be subjected to moderately strong ground shacking, and structures assigned to SDC D, E, or F may be subjected to strong ground shacking. And it is the intent of Committee 318 that the seismic-force-resisting system of structural concrete buildings assigned to SDC D, E or F be provided be special moment frames, special structural walls, or a combination of the two. The ACI 318-08 also presented correlation table between ACI 318-08 and other codes with UBC-1997 [9] about seismic design categories and seismic zones (Table 2.2). Table also shows comparison between 3 codes ACI 318-08, EN 1998-1:2004 and TCXDVN 375:2006 Part 1 on seismic design category and seismicity regions.

Table 2.4 also describes diagram of seismic zonation map in three codes and standards of Social Republic of Vietnam, United States of America, and People's Republic of China (Su [20], Tsang [21]). China's seismic map is shown in Figure 2.22 (Tsang [21]) for return period 475 years (more than a 10% probability of exceedance in 50 years). The seismicity map of the United States as shown in Figure 2.23 (Lorant [22]), UBC-1997 seismic provisions contain six seismic zones, ranging from 0 to 4, equivalents to peak ground acceleration ranging from 0 to 0.4g. Figure 2.24 (McCue [23]) shows seismic hazard map for the Australia, South Pacific and Southeast Asia region with peak ground acceleration for an exceeded probability of 10% within 50 years (in other words, for a return period of 500 years). Figure 2.25 (Solomos et al. [24]) also shows European-Mediterranean seismic hazard map for the peak ground acceleration with 10% probability of exceedance in 50 years under stiff soil condition.

It seems to be that provisions for Vietnamese seismic region classification based on EN 1998-1:2004 are higher than other codes and practising earthquake in Vietnam.

Some countries of Southeast Asia like Singapore, Malaysia, and including Indonesia except Sumatra Island, are only located in low-to-moderate seismicity regions. Additionally, Table 2.3 showed clearly about problem: SDC D, E, F (ACI 318-08) equivalents to DCM, DCH (TCXDVN 375:2006 Part 1) and SCD D, E, F also corresponded Seismic zone 3, 4 (UBC-1997) with PGA (peak ground acceleration) ≥ 0.3g. It means that the current Vietnamese seismic design code should be considered carefully in future. In Vietnam, author would like to recommend using $0.05g \leq a_g < 0.2g$ for low-to-moderate seismicity regions to replace current equivalent provisions in TCXDVN 375:2006-Part 1 as shown in Table 2.5. It leads to the use of ductility class low (DCL) for seismic design and detail in reinforced concrete buildings is allowed in low-to-moderate seismicity regions for many sites in Vietnam (Table 2.6). According to current seismic code, too many regions are located in strong seismicity regions by requirement of $a_g \geq 0.08g$. Table 2.7 will summaries of correlation between standards and codes of United Stated of America, Europe and Vietnam on seismic design requirements to seismic zones and gives recommendation on current Vietnam's standard. Obviously, two proposed solutions seem to meet some codes like P.R.China, United States of America, some countries in Southeast Asia, and not only for current practice on earthquake, but also for Vietnam's history on practising earthquakes. This proposed recommendation will be illustrated and proved clearly and fully in next chapters on problems of wide beam-column connections and transfer structures, transfer beams in Vietnam.

CHAPTER III

WIDE BEAM-COLUMN JOINTS AND BEAM-CORE WALL JOINTS

3.1 EN 1998-1:2004

For the development length: EN 1992-1-1:2004 [10], Section 8 applies in EN 1998-1:2004 [4], the design anchorage length of longitudinal reinforcement, l_{bd}, is (Clause 8.4.4(1)):

$$l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ l_{b,rqd} \geq l_{b,min} \tag{3-1}$$

The basic required anchorage length, $l_{b,rqd}$, for anchoring the force $A_s\sigma_{sd}$ in a straight bar assuming constant bond stress equal to f_{bd} follows from:

$$l_{b,rqd} = (\phi/4)\ (\sigma_{sd}/f_{bd}) \tag{3-2}$$

The design value of the ultimate bond stress, f_{bd}, for ribbed bars, is:

$$f_{bd} = 2.25\eta_1\eta_2 f_{ctd} \tag{3-3}$$

The minimum anchorage length if no other limitation is applied, $l_{b,min}$, are:

- For anchorages in tension:

$$l_{b,min} > \max\{0.3l_{b,rqd};\ 10\phi;\ 100\text{ mm}\} \tag{3-4}$$

- For anchorages in compression:

$$l_{b,min} > \max\{0.6l_{b,rqd};\ 10\phi;\ 100\text{ mm}\} \tag{3-5}$$

where: α_1, α_2, α_3, α_4 and α_5 are coefficients given in table in EN 1992-1-1:2004

α_1 is for the effect of the form of the bars assuming adequate cover.

α_2 is for the effect of concrete minimum cover.

α_3 is for the effect of confinement by transverse reinforcement

α_4 is for the influence of one or more welded transverse bars (ϕ_t>0.6ϕ) along the design anchorage length l_{bd}.

α_5 is for the effect of the pressure transverse to the plane of splitting along the design anchorage length.

σ_{sd} is the design stress of the bar at the position from where the anchorage is measured from.

f_{ctd} is the design value of concrete tensile strength.

η_1 is a coefficient related to the quality of the bond condition and the position of the bar during concreting.

η_2 is related to the bar diameter.

φ is diameter of a reinforcing.

For the detailing of anchorage of reinforcement in the earthquake resistance design, EN 1992-1-1:2004, Section 8, with the additional rules of the following below clauses apply (EN 1998-1:2004, Clause 5.6.1(1)P).

About geometric constraints, Clause 5.4.1.2.1(3)P is "To take advantage of the favourable effect of column compression on the bond of horizontal bars passing through the joint, the width b_w of a primary seismic beam shall satisfy the following expression:

$$b_w \leq \min\{b_c + h_w; 2b_c\} \tag{3-6}$$

where h_w is the depth of the beam and b_c is the largest cross-sectional dimension of the column normal to the longitudinal axis of the beam."

According to Equation (3-6), the beam width can be larger than the column section, twice the column dimension, it is used to design for both DCM, DCH (5.5.1.2.1(5)P). Regardless of anchorage of reinforcement, Clause 5.6.2.2(1)P is "The part of beam longitudinal reinforcement bent in joints for anchorage shall always be placed inside the corresponding column hoops."

EN 1998-1:2004 indicates three measures if the development length cannot satisfied in exterior beam-column joints because the depth of the column parallel to the bars is too shallow, to ensure anchorage of the longitudinal reinforcement of beams:

a) The beam or slab may be extended horizontally in the form of exterior stubs (Figure 3.1.a).

b) Headed bars or anchorage plates welded to the end of the bars may be used (Figure 3.1.b).

c) Bends with a minimum length of $10d_{bl}$ and transverse reinforcement placed tightly inside the bend of the bars may be added (Figure 3.1.c).

For the beam-core wall joints in Figure 2.7.b, 2.7.c, the Figure 3.2, 3.3 indicate the some cases of detailing for the wall provided in EN 1998-1:2004. Therefore, joints in Figure 2.7 can be computed in accordance with provisions in EN 1998-1:2004. The width of core wall in Figure 2.7.b is seem to be too small to support two wide beams at connection and not enough space to detail reinforcement for both the wall and the wide beam. However, core wall in Figure 2.7.a, 2.7.c has been detailed as a column at the corner and at the end of the wall, but only half part of the wide beam is inside column section in Figure 2.7.b. The joint in Figure 2.7.c is likely same the beam-column corner joint in Figure 2.4.c, except not the wide beam in other direction.

Regardless of effective joint width, Clause 5.5.2.3(1)P requires "The horizontal shear acting on the core of a joint between primary seismic beams and columns shall be determined taking into account the most adverse conditions under seismic actions, i.e. capacity design conditions for the beams framing into the joint and the lowest compatible values of shear forces in the other framing elements." For the exterior beam-column joints, simplified expressions for the horizontal shear force acting on the concrete core of the joints, V_{jhd}, may be used as follows:

$$V_{jhd} = \gamma_{Rd} A_{s1} f_{yd} - V_C \quad (3\text{-}7)$$

where: A_{s1} is the area of the beam top reinforcement;

V_C is the shear force in the column above the joint, from the analysis in the seismic design situation;

γ_{Rd} is a factor to account for overstrength due to steel strain-hardening and should be not less than 1.2.

Clause 5.5.3.3(1)P requires "The diagonal compression induced in the joint by the diagonal strut mechanism shall not exceed the compressive strength of concrete in the presence of transverse tensile strains", in the absence of a more precise model, this requirement may be satisfied by means of the subsequent rule: for at exterior beam-column joints:

$$V_{jhd} \leq 0.8 \eta f_{cd} b_j h_{jc} \, (1 - \nu_d/\eta)^{1/2} \quad (3\text{-}8)$$

where: η is degree of connection, $\eta = 0.6(1 - f_{ck}/250)$, f_{ck} is given in MPa;

h_{jc} is the distance between extreme layers of column reinforcement;

ν_d is the normalized axial force in the column above the joint.

b_j is the effective joint width, if $b_c < b_w$ (for mentioned case with wide beam):

$$b_j = \min\,[b_w; (b_c + 0.5h_c)] \quad (3\text{-}9)$$

b_j if $b_c > b_w$: See detail in EN 1998-1:2004 [4].

The problems of switch from rigid to hinge connection (Figure 2.8.b, 2.8.c), this joint becomes connections transmitting shear forces (Clause 10.9.4.4, EN 1992-1-1:2004). For shear transfer in interfaces between two concretes, the shear stress at the interface between concrete cast at different times should also satisfy the provisions 6.2.1 through 6.2.5 in EN 1992-1-1:2004. It is recommended when in the absence of more detailed information surfaces may be classified as very smooth, smooth, rough or indented, c and μ are factors which depend on the roughness of the interface:

- Very smooth: a surface cast against steel, plastic or specially prepared wooden moulds: c=0.25 and μ=0.5.

- Smooth: a slipformed or extruded surface, or a free surface left without further treatment after vibration: c=0.35 and μ=0.6.

- Rough: a surface with at least 3 mm roughness at about 40 mm spacing, achieved by raking, exposing of aggregate or other methods giving an equivalent behaviour: c=0.45 and μ=0.7.

In addition, the anchorage of reinforcement at supports is followed (Clause 10.9.4.7, EN 1992-1-1:2004): Reinforcement in supporting and supported members should be detailed to ensure anchorage in the respective node, allowing for deviations. The effective bearing length a_1 is controlled by a distance d from the edge of the respective elements where (Figure 3.4):

$d_i = c_i + \Delta a_i$ with horizontal loops or otherwise end anchored bars.

$d_i = c_i + \Delta a_i + r_i$ with vertically bent bars.

where: c_i is concrete cover, Δa_i is a deviation, r_i is the bend radius.

3.2. ACI 318-08

In flexural members of special moment frames, geometric constraints of beam is provided in Clause 21.5.1.4: "Width of member, b_w, shall not exceed width of supporting member, c_2, plus a distance on each side of supporting member equal to the smaller of (a) and (b):

(a) Width of supporting member, c_2, and

(b) 0.75 times the overall dimension of supporting member, c_1."

This clause can be expressed in formula is (Figure 3.5):

$$b_w = \min \{c_2 + c_2; c_2 + 0.75c_1\} \tag{3-10}$$

The development length of bars in tension, according to Clause 21.7.5.1, for bar sizes No. 3 (9.52mm) through No. 11 (35.81mm), the development length, l_{dh}, for a bar with a standard 90-degree hook (90-degree hook shall be located within the confined core of a column or of a boundary element) in normalweight concrete shall not be less than the largest of $8d_b$, 6 in., and the length required by equation (in.):

$$\ell_{dh} = f_y d_b / (65 \sqrt{f'_c}) \tag{3-11}$$

Regardless of the transverse reinforcement in joints, Clause 21.7.3.3 requires "Longitudinal beam reinforcement outside the column core shall be confined by transverse reinforcement passing through the column that satisfies spacing requirements of 21.5.3.2, and requirements of 21.5.3.3 and 21.5.3.6, if such confinement is not provided by a beam framing into the joint.", it refers to a joint where the width of the beam exceeds the corresponding column dimension. In that case, beam reinforcement not confined by the column reinforcement should be provided lateral support either by a beam framing into the same joint or by the transverse reinforcement given in Figure 3.5.

For the beam-core wall joints in Figure 2.7.b, 2.8.c, the beam-core wall joints are similar beam-column joints but at the positions of connection then the core wall has to be designed and detailed as a boundary elements with relevant provisions in ACI 318-08, such as the need for special boundary elements at the edges of wall shall be designed in accordance with Clause 21.9.6, and the boundary element transverse reinforcement shall be satisfied Clause 21.9.6.4, 21.9.6.5.

Relating to the effective joint width, Clause 21.7.4.1 and Figure 3.6 indicate: The effective joint width, b_j, shall not exceed the smaller of (a) Beam width plus joint height, (b) Twice the smaller perpendicular distance from longitudinal axis of beam to column side:

$$b_j = \min [(b + h); (b + 2x)] \tag{3-12}$$

Nominal shear strength V_n of the joint for normalweight concrete:

(a) For joints confined on all four faces: $V_n \leq 20A_j(f'_c)^{1/2}$

(b) For joints confined on three faces or on two opposite faces: $V_n \leq 15A_j(f'_c)^{1/2}$

(c) For other joints $V_n \leq 12A_j(f'_c)^{1/2}$

For the solutions to solve the joint of beam-core wall from rigid connection to hinge connection in Figure 2.8.b, 2.8.c, the provisions (Clause 16.6-Connection and bearing design) in ACI 318-08 require: Forces shall be permitted to be transferred between members by grouted joints, shear keys, mechanical connectors, reinforcing steel connections, reinforced topping, or a combination of these means. The adequacy of connections to transfer forces between members shall be determined by analysis or by test. Where shear is the primary result of imposed loading, it shall be permitted to use the provisions of deep beams as applicable. The allowable bearing stress at the contact surface between supported and supporting members and between any intermediate bearing elements shall not exceed the bearing strength for either surface or the bearing element, or both. Unless shown by test or analysis that performance will not be impaired, (a) and (b) shall be met (Figure 3.7):

(a) Each member and its supporting system shall have design dimensions selected so that, after consideration of tolerances, the distance from the edge of the support to the end of the precast member in the direction of the span is at least $l_n/180$, but not less than:

For solid or hollow-core slabs: 2 in.

For beams or stemmed members: 3 in.

(b) Bearing pads at unarmored edges shall be set back a minimum of 1/2 in. from the face of the support, or at least the chamfer dimension at chamfered edges.

3.3. Discussion

Firstly, Table 3.1 shows key concepts on design and detail of structures for earthquake resistance according to codes of Vietnam, Europe and United States of America. All major important concepts on seismic design and detail did not exist in Vietnam standards until before 2006. As regards to beam-column joint design, some countries in regions of low-to-moderate seismicity ignored this problem, such as England, some city and countries adopted or used British standard (BS) like Hong Kong, Singapore, Malaysia, etc. EN 1992-1-1:2004 used in very low and low seismicity regions (for non-seismic members) also are allowed to ignore beam-column joints. Vietnam adopted EN 1998-1:2004 as national standard from 2006, almost structures consequently are not accordance with current seismic standard. In order to evaluate safety of beam-column joints constructed, Wong [25] presented on seismic behaviour of reinforced concrete beam-column joints, which are not-seismically designed, in regions of moderate seismicity.

About wide beam-column joints, according to Benavent-Climent [26], ACI 352R-91 recommended that wide beam connections not be used in structures to dissipate energy inelastically in response to earthquake shaking. In 1995, ACI 318-95 permitted the use of wide beam-column connections in design of structures for earthquake resistance, if all wide beam longitudinal reinforcements not passed through or anchored in the column core was properly confined and if the beam width, b_b, was not more than column width, b_c, plus the distances on each side of the column not exceeding $0.75h_b$. For exterior joints, in order to anchor the wide beam longitudinal reinforcement, satisfying the first condition requires transverse beams whose depth h_s is commonly larger than the wide beam width, h_b.

Both EN 1998-1:2004 and ACI 318-08 have requirements in case of wide beam design, but they lack the terms on design and detail, especially for the exterior and corner wide beam-column connections. Clause 5.6.2.2(1)P (EN 1998-1:2004) is "The part of beam longitudinal reinforcement bent in joints for anchorage shall always be placed inside the corresponding column hoops." It maybe means that it shall be arranged transverse reinforcements in the column core which is satisfied code's requirements. Consequently, anchoring of longitudinal reinforcement in wide beam at wide beam-column exterior and corner joints, and joints of beam-core wall (shear wall) are similar to the conventional joints as Figure 2.3 shown, except to require column hoops to confine beam longitudinal reinforcements passing outside the column core and boundary element. Particularly, column hoop of EN 1998-1:2004 is transverse reinforcements in the column core or not. According to the author, column hoop is not transverse reinforcements in the column core (See Figure 3.5 for transverse reinforcement). The Clause 5.6.2.2 and subclause 5.6.2.2(1)P (EN 1998-1:2004) about anchorage of beam reinforcement are not clear because they are not to point out beam reinforcements passed outside of the column core, as well as about detail of column hoop or transverse reinforcements in the column core.

For the development length of longitudinal bars in beam, only EN 1998-1:2004 requires some methods to ensure development length in joints (Figure 3.1), but EN

1998-1:2004 does not have specific requirements for the wide beam. Paulay and Priestley [27] also indicate some measures as EN 1998-1:2004 shown. For the development length of beam longitudinal bars passing outside the column core in exterior and corner beam-column joints, the outside bars is seem to be similar to inside bars, there is no any difference. Spanish seismic code (Benavent-Climent [26]) requires transverse beam height, h_s, is greater than the wide beam height, h_b for anchored provisions. In wide beam-column joints and beam-core wall joints, the development length of anchorage of beam reinforcements bars are about 45ϕ (ϕ is diameter of reinforcement) according to Vietnam's standard (TCXDVN 356:2005 [8] and TCXD 198:1997 [7]) and they are arranged as shown in Figure 3.8. This development length, l_{an}, is quite so long in comparison with about 30ϕ and 20ϕ according to EN 1992-1-1:2004 [10] and ACI 318-08 [2], respectively. Consequently, joints are congested by many reinforcing steels of beam, column or core wall, especially when the development length goes to 45ϕ and more. In this case, ACI 318-08 has the shortest development length.

In case of the development length is not enough because limitation of the wide beam height and the development length is needed to reduce to avoid steel congestion in connections, Chun et al. [28] have been indicated that based on experimental and analytical studies, prior researchers suggested that the development length for headed bars in beam-column joints be equal to approximately 60 to 70% of that for hooked bars. Thus, Clause 4.5.3.3 ACI 352R-02 [29] was revised to allow use of headed bars embedded into a joint with a distance equal to 75% of the development length for a standard 90-degree hooked bar.

The problems of design and detailing for exterior and corner wide beam-column joints with beam longitudinal reinforcement passing outside the column core are provided in ACI 318-08, but it is not clear for EN 1998-1:2004 as mentioned above (about column hoop). However, they only indicate the requirements for transverse reinforcement in the column core. ACI 318-08 have requirement about transverse reinforcements in the column core but it is not full, because how to design and detail of this reinforcement. For calculation of horizontal shear force acting on the concrete core of the joints, EN 1998-1:2004 has provision in case of the wide beam ($b_c<b_w$), while ACI 318-08 has no clear provisions, formula in ACI 318-08 for nominal shear strength of joint is used in case of $b_c>b_w$.

The R21.7.3 in ACI 318-08 also requires that "additional detailing guidance and design recommendations for both interior and exterior wide-beam connections with beam reinforcement passing outside the column core may be found in Reference 21.8." Reference 21.8 relates to ACI 352R-02 and it does not mention here fully (See Table 3.1). In ACI 352R-02, Clause 2.2.1: "These recommendations apply when the design beam width b_b is less than the smaller of $3b_c$ and (b_c+1.5h_c), where b_c and h_c are the column width and depth, respectively." And also according to ACI 352R-02, the limit of b_c is intended to ensure the complete formation of a beam plastic hinge in Type 2 connections. Type 2 is a connection that has members that are required to dissipate energy through reversals of deformation into the inelastic range. Connections in moment-resisting frames designed according to ACI 318-02 Sections 21.2.1.3 and 21.2.1.4 are of this category. According to Moehle et al. [30], this guide is written mainly to clarify requirements of ACI 318-08, but it also introduces other guides such as ACI 352R-02 and it presents other recommendations for good design and construction practices. This guide is written to clearly differentiate between requirements of ACI 318-08 and other recommendations.

In Indian code, Jian et al. [31] indicate that when the width of beam exceeds corresponding column dimension, transverse reinforcement as required by Clause 7.4.7 and 7.4.8 of IS 13920:1993 shall be provided through the joint to provide confinement for longitudinal beam reinforcement outside the column core if such confinement is not provided by a beam framing into the joint. In such a case, the value of width of beam b_b should be less than the values of $3b_c$ and b_c +$1.5h_c$, where b_c and h_c are the column width and depth, respectively. In that case, the beam reinforcement not confined by the column reinforcement should be provided lateral support either by a girder framing into the same joint or by transverse reinforcement. The maximum beam width recommended here is based on some experiments on joints between wide beam and column. Uma and Jian [32] also presented critical review of recommendations of well established codes regarding design and detailing aspects of beam-column joints for ACI 318, NZS 3101 and EN 1998-1. In Spain, according to Benavent-Climent [26], the pre-1994 national seismic code PDS-74 contained no provision for wide beam. The 1994 seismic code NCSE-94 prohibited the use of wide beam in the southern regions of Spain when the design peak ground acceleration is larger than 0.16g. And now, the current Spanish seismic code NCSE-02 permits the use of wide beams in earthquake-prone regions, if transverse beam with $h_s>h_b$ are provided in the exterior connections and if the position of the wide beam longitudinal reinforcements does not exceed b_c plus $0.5h_b$ distances on each side of the column. In addition to the anchorage conditions and geometric limitations, the Spanish current seismic codes prescribe special reinforcement details aimed at attaining some degree of ductility at the wide beam ends. Here is noted about value of 0.16g in old and current Spanish seismic code, Vietnam seismic code, TCXDVN 375:2006-Part 1, requires that design peak ground acceleration $a_g \geq 0.08g$ used for strong seismicity regions.

In wide beam designs, if beam longitudinal bars pass outside of the column core, the diagonal strut also will be outside of the column, with no equilibrating vertical compression at its upper and lower ends. The outer parts of the wide beam would lead to shear off, resulting in early failure. Two cases of possibilities exist to overcome this problem. For wide beam-column joints that the column section is wider than the wide beam width, it is easy to require that all of the wide beam top reinforcements placed within the width of the column core. Otherwise, as the wide beam width is wider than the column section, if wide beam reinforced bars are passed outside the joint, vertical hoops can be provided through the joint region to carry the vertical component of thrust from the compression strut. In extreme but not unusual cases in strong seismic regions, very wide beams are used, several times wider than the column, with beam depth only about 2 times the slab depth. In such cases, a safe basis for joint design is to treat the wide beam as slab and follow the recommendations for slab-column connections (Nilson et al. [33]).

The details of the joints in Figure 2.7.b, 2.7.c is acceptable but they are not safe in the design and the detail. Particularly with the joint of Figure 2.7.c or 2.4.c, 2.4.f, according to ACI 352R-02, Clause 2.2.2: “Eccentric connections having beam bars that pass outside the column core are excluded because of a lack of research data on the anchorage of such bars in Type 2 connections under large load reversals.” With Figure 2.7.b, joint must be paid attention to ensure the development length of the longitudinal bars in core wall to meet the code’s requirements but also to ensure easily in construction process in site. To solve this issues, the need is calculated the problem of core wall, so the volume of calculation will be complicated, but does not resolve the issues for wide beam longitudinal reinforcement ratio that concentrated so much in joint. Therefore, in this position, it is better to expand the core wall section to meet requirements, such as transfer to column member as shown in Figure 2.7.a, then all problems will be clearly and more simpler, and also handle such for joint in Figure 2.8.a instead of forming the hinge joint. Building with the joint of Figure 2.7.c were constructed and put to operation. Now, no unusual anything happens, this could be still safe, but the condition is not strong enough earthquake force acted to structure system.

According to Gentry and Wight [16], when structure subjected to earthquake loading, beam bars anchored outside of the column core may be unable to transfer their tension to the column, either because bond is lost or because the transverse elements transmitting the tension to the column has failed or lost stiffness. It leads formation of an incomplete beam plastic hinge with smaller beam moment strength than the before intended-designer, which in turn reduces the lateral force required to form a collapse mechanism for reinforced concrete frame. Due to these concerns, it has been recommended that the wide beam-column joints not to be used as a part of a ductile moment resisting reinforced concrete frame. The experimental tests carried out and summarized by Gentry and Wight [16] shows that the wide beam-column exterior and interior connections can be used in strong seismic zones if they are detailed correctly. If they are not detailed correctly, the exterior joints will be incapable of transferring the plastic hinge bending moments to the column because of the transverse beam cracks in torsion. To prevent this cracking of the transverse beam, limits on the torsion applied to the transverse beam are proposed.

Hatamoto et al. [34] shows that in the design of the wide beam-column frames as ductile moment resisting frames, the following recommendations are made: The beam to column width ratio should be less than two. The amount of the wide beam longitudinal reinforcements passed outside of the column core should be limited in term of torsion stress in the outside beam region. Sufficient transverse reinforcement should be provided in the outside beam region not only to improve the torsion rigidity after cracking but also to provide adequate anchorage for the beam longitudinal reinforcements passed outside of the column core. Testing result assessment (Popov et al. [35]) indicates that placement of some of the wide beam longitudinal bars outside of the column core is permissible and leading to lesser congestion of reinforcing steel in connections. The wide beam longitudinal bars outside of the column core, as well as the slab bars, are strained significantly to warrant their consideration as part of the lateral resisting mechanism of the joint.

Kulkarni and Li [36], [37] have been presented experimental and finite element numerical investigations on interior and exterior wide beam-column joints. Especially, some recommendations about exterior and interior wide beam-column connections in existing reinforced concrete frames subjected to earthquake loading proposed by Benavent-Climent et al. [38], [39]. According to Kulkarni and Li [36], the potential advantages and applications of the wide beam systems in a lateral load resisting structure system are often ignored due to the lack of understanding of its seismic performance. Design code BS 8110-1997 strictly restricts the design of wide beam-column joints to resist earthquake loads. Moreover, some geometric restrictions on the elements of wide beam-column connections are often imposed based on historic design practices. However, Stehle et al. [40] and Siah et al. [41] found that by incorporating a special type of bars detailing, no beam width limitations are required for the design of wide beam flooring systems in strong seismicity regions. Using wide beam connections for primary lateral load resisting system depend on level of seismicity. These interior wide beam-column connections could be suitable for use in strong seismicity regions as part of a moment-resisting frame. Among previous studies, the primary concern for wide beam-column joints is the effectiveness of the longitudinal reinforcements pass outside of the column core because of the different load transfer mechanism. Hatamoto et al. [34] indicated that the amount of beam reinforcement placed outside of the column core should be limited to reduce torsion stress. In addition, sufficient confinement should be provided to the outermost part of beam to improve the torsion rigidity and provide adequate anchorage for the beam reinforcement. Some investigations on wide beam-column joints also focused on lower stiffness, energy dissipation capacity, and bond slip of column reinforcements (Ehsani and Wight, 1985; LaFave and Wight [42]). Recently, a series of research projects conducted by LaFave and Wight (2001) has highlighted the impact of these parameters on the behavior of wide beam-column joints. Experimental test results showed that the energy dissipating capacity of wide beam-column joints is almost equal to conventional beam-column connection (LaFave and Wight, 1999 [42], 2001). Until now, experimental research on wide beam-column joints was not sufficient enough to fully understand their seismic behavior, while numerical investigations in these areas are very scarce. In finite element numerical study, 3D model developed is validated by comparing analysis results with experimental test results, which has shown a good agreement.

The both codes of EN 1998-1:2004 and ACI 318-08 have provisions on the wide beam width, but it should be more regulations about number of longitudinal reinforcements outside the column core, the development length of these bars, and give instructions on cases that the development length is not enough because limitation of the wide beam height. According to Kulkarni and Li [36], Paulay et al. (1978) recommended that at least three-fourths of the beam longitudinal bars should pass through the column core. About the effective wide beam width, it is controlled by the torsion strength of the transverse beam, so that transverse beam is a critical issue in the design of wide beam connections, which needs to be designed and detailed carefully. In the wide beam-column joints, when signed with suitable parameters, perform quite well in carrying the horizontal lateral loads as they can generally attain their strength and deformation capacity (Li and Kulkarni [37]).

There is different view on the wide beam width: According to ACI 318-08, it depends only on column section, while in EN 1998-1:2004, it depends on column section and also wide beam height. According to Benavent-Climent et al. [38], concerns about the behavior of the wide beam-column connections under lateral loads led codes to prohibit or limit their use in seismic regions, in United State of America, ACI 352R-02: $b_b \leq \min\{b_c+1.5h_c; 3b_c\}$, in New Zealand, NZS 3101-95: $b_b \leq \min\{b_c+0.5h_c; 2b_c\}$, in European code EN 1998-1:2004: $b_w \leq \min\{b_c+h_w; 2b_c\}$, and in Spain, NCSE-02: $b_b \leq (b_c+h_b)$. Regardless of stirrup for the wide beam, Figure 3.6 presents stirrup configurations in wide beam [43] that satisfies ACI 318-08.

In design of wide beam-column connections, it is needed to learn cases from other countries like Spain, Italy… According to Benavent-Climent [26], most of the reinforced concrete moment-resistant frames with wide beam-column connections located in countries of the earthquake-prone Mediterranean area were built before its limitations and disadvantages became be evident, and before the provisions of above-mentioned seismic code became compulsory. Consequently, this structure shares the following features: (i) the wide beam width substantially exceed the geometry constrains provided by the current seismic codes; (ii) the almost wide beam longitudinal bars does not anchor directly into the column core; (iii) there are no transverse beams or special reinforcement to resist torsion moment in the lateral zones of the wide beam adjacent to the column. This torsion is generated by the wide beam longitudinal bars located at a distance from the column’s side faces. Further, the connections also do not satisfy modern seismic provisions on necessary ductility level, because (a) the columns may be weaker than the adjacent beams when longitudinal bar greater than that required by analysis is provided to the beams to control the vertical deformations due to gravity loading and to reduce the required anchored development length; (b) there are no transverse hoops within the joint to carry the joint shear; (c) the transverse reinforcement at the beam and column ends may be insufficient to resist the shear associated with the maximum feasible flexural strength at these sections; (d) the lap splices in the column's flexural reinforcement are located just above the floor levels. All above problems are similar to Vietnam not only in the past, but also in the future due to many standards are still not consistent in Vietnam, and it takes several years for architects and structural engineers, as well as owners to understand.

Secondly, in beam-core wall joints, when the joint detail is rigid connection, the beam longitudinal bars are too much in top and bottom layers, not good clearly for construction and computation of this type. The transfer from rigid joint to hinge joint to overcome these disadvantages, but have difficulties to construct the core wall with corbel to support the wide beam, and how to consider the vertical vibration, it is not acceptable solution. Moreover, bearings shall be designed and detailed to ensure correct positioning, taking into account production and assembling deviations, especially for the complicated factors which depend on the roughness of the interface. In reality, after discuss between the design engineers and evaluation engineers, structural designers has been took the solution of rigid connection. To detailing the hinge joints, can also use the some solutions relate to other anchorage types of longitudinal bars in the wide beam. In order to solve problems of beam-core wall joints mentioned in Chapter II, Figure 2.7.b and 2.7.c, the first solution must be expanded more section of core wall to ensure the principles of strong-column and weak-beam, especially for cases of primary seismic beams. If using only reinforcement solution for local position of core wall linked beam as boundary element then maybe not enough. For the position can not expand area of core wall, they must be based on calculations to meet with design and detail requirements. The detail of the transverse reinforcement in boundary elements shall be followed the requirements of the two codes. The provisions for boundary elements in EN 1998-1:2004 are more detail and clear than in ACI 318-08.

When assigning hinge connections in structure system, especially at the core wall, it is needed to considered mechanism of lateral force transfer direction to avoid less lateral stiffness by hinge joints. Moreover, the core wall is major element to receive all lateral loadings, therefore it is careful to put hinge connections at the core wall. It is very wasted when hinge connections detailed at the core wall. Generally, in high rise buildings subjected to earthquake loading, the detail of hinge joints for the beam-core wall connections may not be the right solution. There are many problems when using hinge connection, such as force transfer mechanism, flexibility of structure, serviceability of building. For both EN 1998-1:2004 and ACI 318-08, the details of bearing (hinge connection) are not to be provisions or additional provisions in part of the earthquake-resistant structures, so that hinge connection shall not be allowed to use in seismicity zones. The switch from rigid to hinge connection to reduce wide beam longitudinal reinforcement ratio in joint leads to reduce the stiffness of overall structural system, and thus violated the terms for design for earthquake resistance, with special moment frames in ACI 318-08 and DCM, DCH in EN 1998-1:2004 that mentioned in Chapter II. The hinge connection at the core wall is possible but it is used only for non-earthquake members (secondary seismic members) in buildings in very low to low seismicity cases as shown in Table 3.4.

Regardless of boundary elements, the seismic provisions of ACI 318-08 also offer two methods for determining the need for special boundary elements. The first method (Clause 21.9.6.3) considers concrete stress, and permits expected maximum concrete compressive stresses to be computed assuming a linear stress distribution over the depth of the section using gross section properties. The second method (Clause 21.8.6.2) considers the location of the neutral axis of a section when subjected to factored loads. When the length of the compression zone exceeds a critical value, special boundary elements are required (Fortney and Shahrooz [44]).

Lastly, about using wide beam-column connections in seismic regions and also Vietnam, according to Benavent-Climent et al. [38], wide beam connections has been widespread over the past three decades in countries of the moderate-seismicity Mediterranean area, like Spain or Italy, as a lateral force resisting system. Almost existing structures do not satisfy the requirements of current seismic codes, and thus their safety in the event of a severe earthquake is a matter of great concern among structural designers and researchers. Maybe these problems are similar to problems that Vietnam faced.

Table 3.2 shows brief summary of design and detail of the wide beam-column joints, as well as beam-core wall joints in some different standards and codes in Vietnam, Europe, and United States of America. ACI 352R-02 also presents two examples of the wide beam-column interior and exterior connections corresponding Figure 2.4.a and Figure 2.4.c, but transverse beam in example of exterior connection is not the wide beam. Before TCXDVN 375:2006-Part 1 is official in Vietnam, concept of design of beam-column joints as shear strength (V_u or V_{jhd}) of joint did not exist in Vietnam's design standards, as well as concepts of lateral-resisting moment frame, ductility level, strong-column and weak-beam (Table 3.1). After TCXDVN 375:2006-Part 1 adopted, some existing projects in Hanoi, Ho Chi Minh city and others provinces are faced problems with wide-beam connections, because these cities are located in regions of strong seismicity, while before that, they were located in low seismic regions (Table 3.3). According to TCXDVN 375:2006-Part 1, the design of wide beam-column joints could be applied in few some zones in Ho Chi Minh and Da Nang city, where located in very low to low seismicity regions, but it could not be used in Hanoi capital, Hai Phong city located in strong seismicity regions, etc. However, as mentioned in Chapter II, section 2.4, the key problem here is that classification of value for peak ground acceleration, a_g, corresponding strong seismicity regions in current Vietnam's standard. The author has proposal for changing this value (Table 2.6, 2.7). Moreover, Vietnam's earthquake situation is not quite different from some Southeast Asia countries located in low-to-moderate sesimicity regions, like Malaysia, Singapore, Thailand, large regions in India, or Hong Kong; Spain, Italy and some other countries in Mediterranean zone, where wide beam constructions had been done over the past year according to British standard (BS), Indian code, Spanish standard, respectively.

Additionally, experimental tests had been done to indicate that average drift ratio ranges from 1% to 2%, average displacement ductility ratio in range from 3 to 4 and even more, this ductility is moderate and higher, as shown in Table 3.5 and Figure 3.10. Limitations of drift ratio are 0.5%, 0.5%, and 2.0% according to EN 1998-1:2004, 4.4.3.2, ACI 318-08, 21.13.6, and UBC 1997 Volume 2, 1630.10.2, respectively. According to EN 1998-1:2004, design criteria on local ductility condition is that for the required overall ductility of the structure to be achieved, the potential regions for plastic hinge formation shall possess high plastic rotational capacities (Clause 5.2.3.4(1)P), and displacement ductility factor shall be greater than at least about 4 using for strong seismicity regions where $a_g \geq 0.08g$ corresponding to DCM, DCH (Clause 5.2.2.2). Therefore, this joint type could be considered to use in regions of low-to-moderate seismicity where a_g<0.08g. In high and moderate seismicity zones, Table 3.6 indicates that joint shear strength for some experimental tests do not satisfy code's requirement of EN 1998-1:2004 for design for DCH as controlled formula: $V_{jhd} < V_n$. These specimens and other specimens were designed and detailed according to ACI 318 and ACI 352R codes so that it will be considered more carefully under EN 1998-1:2004. However, EN 1998-1:2004 has two criteria for design of beam-column joints: Design for DCM (Clause 5.4) and design for DCH (Clause 5.5), in which it is only to check shear strength for beam-column joints in design for DCH (Clause 5.5.3.3), as given in Table 3.2. While, there are only additional detail requirements for design for DCM (Clause 5.4.3.3). Since, it can be concluded that wide beam-column joints can be used in regions of moderate seismicity corresponding design for DCL and DCM according to EN 1998-1:2004.

Most of the papers recommended that wide beam-column connections could be used in low-to-moderate seismicity regions, even in high seismicity regions with some additional requirements. Brief summary on experimental tests from researchers for wide beam-column joints is given in Table 3.7. In Vietnam, based on experimental tests with displacement ductility ratio is moderate or higher as mentioned above, design of wide beam-column joints could be adopted in low-to-moderate seismicity regions corresponding requirement of design for DCL, DCM according to EN 1998-1:2004 (TCXDVN 375:20060-Part 1) as shown in Table 2.7, this recommendation is conservative.

3.4 Brief summary

(1) For the wide beam-column connections, EN 1998-1:2004 has provisions about shear strength design in the wide beam-column joints (Clause 5.5.2.3, 5.5.3.3),

and Clause 5.4.1.2.1(3)P about geometric constraints with expression of the wide beam width, but it has no specific provisions for detail of the wide beam-column joints. ACI 318-08 has no any provisions about shear strength design, provisions of the wide-beam is only Clause 21.5.1.4 about geometric constraints with expression of the wide beam width.

Thus can be seen that the two codes (EN 1998-1:2004 and ACI 318-08) have some provisions for the wide beams and to mention the beam width is wider the column section according to earthquake resistance design. However, the two codes require to add more detailed provisions on design and anchorage detail of wide beam longitudinal reinforcements passing outside the column core, especially for exterior and corner wide beam-column connections and beam-core wall joints as mentioned on. Of course, the roof-exterior and roof-corner joints will be done, respectively.

Many experimental tests had been done for wide beam-column interior joints, so that this type should be designed in seismic zones, but it depends on each national code. Particularly, with joints as shown in Figure 2.4.b, 2.4.c, 2.4.e, 2.4.f, and Figure 2.7.c (eccentric joints or exterior joints), despite quite many experimental tests, except there is few tests for corner joints, and recommendations in using the wide beam-column joints in seismic regions, but the ACI 352R-02 said not enough research data on the anchorage of wide beam longitudinal bars outside the column core. Since, the need for performing experiment tests to obtain empirical conclusions for this cases. However, it is recommended for joint of Type 2. For Type 1-no earthquake load, the design and detail for eccentric joints in the wide beam can be accepted. Since, the design and detailing of eccentric joints shall be considered carefully in seismic zones, especially strong seismicity regions, the safe solution will be detailed column dimensions at least equal to the wide beam width, or detailed as column or shear wall with shape of T and L at exterior and corner joints, respectively. These columns or shear walls will have section sizes, especially for width of shear wall, are enough in supporting the wide beam. In design of the wide-beam column joints, it is considered to use transverse beam for anchorage of wide beam longitudinal bars.

The anchoring of longitudinal reinforcement in the wide beam at beam-column exterior and corner joints, and joints of beam-core wall (shear wall) are similar to the conventional joints, except to provide transverse reinforcement through the column core to confine beam longitudinal reinforcements passing outside the column core and boundary element. This hoops is also provided in wide beam plastic hinge region according to ACI 352R-02. About design of the development length, it is recommended to use provisions of ACI 318-08 by this is the shortest length to lead to reduce ratio of reinforced steels in connections.

In order to ensure safe and rational factors in design, the provisions for member’s detail should be put first priority, for example, to have to select the member’s size, the ratio between the member’s size, especially at joints, to meet code’s requirements, then to calculate the members, this calculation is only secondary, as may have been clear that the selected structural members meet the requirements for detail then will to meet checking requirements. The computation explanation of members is only for checking of safety level so that they can be adjusted to fit the economic requirements.

Subjective assessment that the codes in developed countries have higher and more detail requirements for earthquake resistance compared to developing countries and less developing countries. This can be derived from scientific research and experimental tests, investigations, surveys, advanced technology, and higher living level on people in developed countries.

(2) For the beam-core wall joints mentioned above, according to the requirements of design of structures for earthquakes resistance, it shall be not calculated and detailed joints as the hinge connections for primary seismic members. For core wall members, it shall be designed and detailed to meet the requirements in supporting the beam and wide beam about control for strength and detailing at connections.

(3) Finally, the wide beam-column connections could be designed in low-to-moderate seismicity zones, even in strong seismicity regions. Some countries in low-to-moderate seismicity regions are allowed to use the wide-beam connections as above-mentioned studies. Some researchers indicated that this connection could be applied even in high seismicity regions. In Vietnam, in order to using wide beam-column connections, the first need shall be changed ranging of peak ground acceleration for low seismicity regions in TCXDVN 375:2006-Part 1, replacing $0.04g \le a_g < 0.08g$ by $0.05g \le a_g < 0.2g$, or even $0.05g \le a_g < 0.15g$ for more conservative if it is necessary, then all structures located in low or low-to-moderate seismicity regions shall be designed with ductility class low (DCL) or corresponding to SDC C (ACI 318-08) in seismic zones of 2A, 2B (UBC 1997), as shown in Table 2.5, 2.6, 2.7. The ACI 352R-02 also said that Type 1-no earthquake load, the design and detail for eccentric joints in the wide beam can be accepted. Since, current seismic codes are only required to add some specific requirements for wide beam-column connections in low-to-moderate seismicity regions.

CHAPTER IV

COUPLING BEAMS

4.1 EN 1998-1:2004

Coupling beams shall be designed and detailed in accordance with Clause 5.5.3.5:

• The coupling of walls by means of slabs shall not be taken into account, as it is not effective.

• The provisions for beam may only be applied to the coupling beams, if either one of the following conditions is fulfilled:

a) Cracking in both diagonal directions is unlikely. An acceptable application rule is:

$$V_{Ed} \leq f_{ctd} b_w d \tag{4-1}$$

where f_{ctd} is the design value of the concrete tensile strength.

b) A prevailing flexural mode of failure is ensured. An acceptable application rule is: $l/h \geq 3$.

• If neither of the conditions in two above condition (a) and (b) is met, the resistance to seismic actions should be provided by reinforcement arranged along both diagonals of the beam, in accordance with the following (Figure 4.1):

a) It should be ensured that the following expression is satisfied:

$$V_{Ed} \leq 2A_{si} f_{yd} \sin\alpha \tag{4-2}$$

where:

V_{Ed} is the design shear force in the coupling element ($V_{Ed} = 2M_{Ed}/l$);

A_{si} is the total area of steel bars in each diagonal direction;

α is the angle between the diagonal bars and the axis of the beam.

b) The diagonal reinforcement should be arranged in column-like elements with side lengths at least equal to $0.5b_w$; its anchorage length should be 50% greater than that required by EN 1992-1-1:2004.

c) Hoops should be provided around these column-like elements to prevent buckling of the longitudinal bars. The provisions of column hoops apply for the hoops.

d) Longitudinal and transverse reinforcement should be provided on both lateral faces of the beam, meeting the minimum requirements specified in EN 1992-1-1:2004 for deep beams. The longitudinal reinforcement should not be anchored in the coupled walls and should only extend into them by 150 mm.

4.2 ACI 318-08

Coupling beams shall be designed and detailed in accordance with Clause 21.9.7:

- The coupling beams with $l_n/h \geq 4$ shall satisfy the requirement of flexural members of special moment frames.

- The coupling beams with $l_n/h < 2$ and with $V_u > 4\lambda A_{cw}(f'_c)^{1/2}$ shall be reinforced with two intersecting group of diagonally placed bars symmetrical about the mid-span.

- The coupling beams not governed by two above categories shall be permitted to be reinforced either with two intersecting groups of diagonally placed bars symmetrical about mid-span or according to flexural members of special moment frames with satisfied requirements for longitudinal reinforcement, transverse reinforcement and shear strength requirements.

- The coupling beams reinforced with two intersecting group of diagonally placed bars symmetrical about the mid-span shall satisfy (a), (b), and either (c) or (d):

(a) V_n shall be determined by: $V_n = 2A_{vd}f_y\sin\alpha \leq 10A_{cw}(f'_c)^{1/2}$ (4-3)

where α is the angle between the diagonal bars and the longitudinal axis of the coupling beams.

(b) Each group of diagonal bars shall consist of a minimum of four bars provided in two or more layers. The diagonal bars shall be embedded into the wall not less than 1.25 times the development length for f_y in tension.

(c) The detail requirements for the coupling beams are described in Figure 4.2, each diagonal element consist of a cage of longitudinal and transverse reinforcement, each cage contains at least four diagonal bars and confines a concrete core.

(d) The detail requirements for the coupling beams are described in Figure 4.3, this is second option for confinement of the diagonals to confine the entire beam cross section instead of confining the individual diagonals. This option can considerably simplify field placement of hoops, which can otherwise be especially challenging where diagonal bars intersect each other or enter the wall boundary.

When the coupling beams are not used as part of the lateral force-resisting system, the requirements for diagonal reinforcement may be waived.

4.3 Discussion

Firstly, in Vietnam standard of TCXD 198:1997 [7], Clause 3.4.2-Detail of shear wall and core wall, there was no any provision about design for the coupling beams, except only sentence of note and figure about detail for the coupling beams as shown in Figure 2.11.b, in which it may be used term of lintel beam for the coupling beams (Table 2.1). Detail of the coupling beams in TCXD 198:1997 also had no any note about diagonal bars, hoops, development length, horizontal and vertical reinforcements, etc, everythings were not clear to design and detail. Since, problems of change for the coupling beams as mentioned above during design and construction process have been occurred for a long time. Until TCXDVN 375:2006 adopted, the coupling beams will be designed and detailed carefully according to standard's requirements (Table 4.1). It is one reason in changing the coupling beams from design stage and then at construction site by designer, contractor, supervisor, owner…

Regardless of the diagonal bar ratio, ACI 318-08 requires at least 4 bars, while EN 1998-1:2004 do not have exact data. For the design and detail of diagonally oriented reinforcement in EN 1998-1:2004, it shall be followed EN 1992-1-1:2004, but for the hoops then its provisions in accordance with EN 1998-1:2004. It means that, the diagonal bars shall be designed and detailed by provisions for the non-earthquake resistant structures in EN 1992-1-1:2004, but the hoops for the diagonal bars shall be followed the earthquake resistant structures in EN 1998-1:2004. Moreover, EN 1998-1:2004 also only have been presented a case for confinement of individual diagonals, and its figure shows not exact for arrangement of diagonal bars, it will be not constructed according to this figure (Figure 4.1) by coincide for the diagonal bars. There is difference between dimensions of group of the diagonal bars, ACI 318-08 requires $b \geq b_w/2$ and $h \geq b_w/5$ (out-to-out dimension), but EN 1998-1:2004 requires both b and $h \geq b_w/2$. Table 4.2 is brief summary on design and detail for the coupling beams based on codes of Vietnam, Europe and United States of America.

According to observations and analysis on experimental tests of conventional coupling beams (monolithic without any slits or keyways), four failure modes can be identified which are: Failure due to diagonal tension or diagonal compression, failure due to shear-slip or due to flexure and shear. It is considered that the weak connection of monolithic coupling beams is the strength against shear. Therefore, placing diagonal bars or additional X-shaped steel in the plastic hinge zones at beam ends was suggested. The ductility of the coupling beams with diagonal bars is good, but because instability of these bars out of its plane may occur, a minimum of thick for core wall or shear wall is required (Dajun et al. [45]). Figure 4.4 (Cheng [46]) shows seven specimens of coupling beams for experimental test, first two specimens, CB1 and CB2, are detailed in accordance with ACI 318-08 option 1 (confinement of individual diagonals) and option 2 (full confinement of beam section), respectively; other specimens are without diagonal bars, with diagonal bars, and three types of discontinuous diagonal bars at mid-span. With design of discontinuous diagonal bars at mid-span, force transfer mechanism through the coupling beams is not occurred effectively, shear wall structures are less stiffness that leads to increase failure possibilities, especially, priority cases of failure and damage will appear in the coupling beams at mid-span as shown in Figure 4.5. Specimen CB3, CB4, CB5 with discontinuous diagonal bars at mid-span prematurely failed at mid-span when testing loads did not reach theoretical capacity, testing shear forces only were about 50% and less of nominal design shears. The displacement ductility ratio of specimens CB3 through CB5 are very small; while displacement ductility ratio of specimen CB2, approximately 6, is larger than CB7 (about 3). Since, specimen CB7 (without diagonal bars, but stirrup arranged to confine all the coupling beams) is used in low-to-moderate seismicity regions, specimen CB2 is option 2 (ACI 318-08) is designed in strong seismicity regions, and specimens CB3 through CB5 are not allowed.

By all above issues, it will be needed larger coupling beam width to have enough space for many reinforcements in the coupling beams, this issue seem to be unreasonable in practice by no engineers like to select large wall width by only reason for code's provisions of the coupling beams. Both codes require cage detailing of diagonal bars, therefore it will need to enough core wall size to place steel cage. The coupling beams width may be selected by reinforcement ratio in the coupling beams. Obviously, the issue of the coupling beams width is not necessarily synonymous with the core wall width. Because of this large wall thickness, economic factor will be not reasonable.

For the ratio l_n/h in the coupling beams, $l_n/h<4$ in ACI 318-08 and $l_n/h<3$ in EN 1998-1:2004, so that EN 1998-1:2004 has more effective than ACI 318-08 by inclination angle of EN 1998-1:2004 is larger than ACI 318-08, that is "Experiments show that diagonally oriented reinforcement is effective only if the bars are placed with a large inclination. Therefore, diagonally reinforced coupling beams are restricted to beams having aspect ratio $l_n/h<4$" (ACI 318-08 R21.9.7). For Figure 5.12 in EN 1998-1:2004 (Figure 4.1 in Chapter IV) also illustrates wrong arrangement for diagonal bars because they shall be staggered as figures in ACI 318-08.

For the longitudinal and transverse reinforcements in the coupling beam cross section, ACI 318-08 requires "Horizontal beam reinforcement at wall does not develop f_y", EN 1998-1:2004 presents "Longitudinal reinforcement should not be anchored in the couple walls and should only extend into them by 150mm". In both codes, these longitudinal and transverse reinforcements are required to meet the minimum requirements and some other ones. In the second case of full confinement in ACI 318-08, the transverse reinforcement has key role to confine coupling beams cross section. In practice, these longitudinal bars will be detailed as longitudinal bars in the wall, it means that they are extended into the wall with long length (see Figure 2.12, Chapter II) by more easy during construction. Consequently, what happen occurs when anchorage them into wall. According to Lequesne et al. [47], fully anchored longitudinal bars are required.

The problem in bending of diagonal bars for enough development length in the wall mentioned in Chapter II (Figure 2.15): The development length of the diagonal bars anchored into the wall piers are usually calculated in tension condition. When this straight development length is not enough then they must be bent with angle as same as beam's longitudinal bars anchored into the column core in beam-column joints. Obviously, the anchored bend of diagonal bars is allowed completely but it is considered bending angle to ensure efficiency and easy to construct, for example, the bend angle should not be less than 90 degree. Both ACI 318-08 and EN 1998-1:2004 have no guidance for bending the diagonal bars when core wall section is shallow. Probably, all experimental tests or research on coupling beams have been carried out by Dajun et al. [45] Harries et al. [48], Fortney [17], Fortney el al. [49] and even more early or lately, all of them did not mention bending angle of diagonal bars.

In order to improve the ductility of the coupling beams and prevent the brittle failure, many other studies of the coupling beams with the X-shaped steel, the coupling beams with a through-slit along the middle depth over the entire beam length, the coupling beams of this type, but also reinforced the ends of two small beams divided by a slit with X-shaped steel bars to strengthen the plastic zone against shear and to prevent shear-slip failure of the shear-compression zone along the normal cracks, and the coupling beams with slits and keyways as shown in Figure 4.6 (Dajun et al. [45]). Cheng [50] researched on steel plate reinforced concrete coupling beams and it showed that the shear strength and ductility of ordinary reinforced concrete coupling beams could be significantly enhanced by using steel plate in replacing of conventional web reinforcements to resist very high shear stresses. Steel fibre reinforced concrete coupling beams tested by Baczkowsk [51]. Addition of steel fibres significantly increases the inclination of the cracking angle and improves the capacity of energy dissipation, changes failure mode from very brittle to more ductile, which is a very important characteristic under earthquake loading. Steel fibre reinforced concrete should be treated as an addition to the concrete that improves its performance under the seismic loadings, especially so in moderate seismicity regions.

In high rise buildings, the coupling beams are necessary element in lateral force resisting structural system. However, in some specify cases, if analytical results show that design stiffness of structure system is enough to satisfy code's requirements then the coupling beams can be dropped as this have more benefits to construct easily, for instance, the coupling beams should be replaced by slab or normal beam. All these will be depended on capacity of structural engineers. But there are some more important issues to consider carefully: Fracture failure at link between slab and core wall, punching condition, and shear force; and Clause 5.5.3.5(1)P said that "Coupling of walls by means of slabs shall not be taken into account, as it is not effective." (EN 1998-1:2004).

The next future testing and studying for the coupling beams:

- Carrying out experimental tests for the coupling beams with difference bend angle of the diagonal bars.

- To continue to review the coupling beams with following solutions: Diagonally reinforced concrete coupling beams, steel coupling beams, concrete-steel composite, steel plate and steel firbe reinforced concrete coupling beams, excluding problems have been done by Dajun et al. [45], Harries et al. [48], Fortney [17], Forney et al. [49], Cheng [50], Baczkowsk [51] and some other researchers.

- Continuing research to propose typical coupling beams in buildings where size of the coupling beams is often fixed and less the changes.

- All research on the coupling beams will be considered to ensure to construct easily, increase quality for important structure resist lateral loads.

4.4 Brief summary

(1) Necessitating the use of diagonal bars in the coupling beams, detailing diagonal bars and changing content of diagonal bars in the coupling beams: No changes of detail for the diagonal bars are permitted, all changes or modifications are violated the terms of both codes.

The experimental studies and codes have been specified the reinforcement of diagonal bars in the coupling beams is needed, but the steel congestion in the coupling beams is the problem needed to consider further in future research to bring the best performance during construction.

(2) Discontinuity diagonal bars at mid-span: In the design or construction for the coupling beams, the diagonal bars do not extend to intersect at mid-span (Figure 2.12.b, 2.13.b) or diagonal bars is not provided (Figure 2.12.a): According to codes, as well as experimental tests have been carried out by Fortney [17], Fortney et al. [49], Cheng [46], these details shall be not acceptable. It is only used for the cases of small opening hole, and for the coupling beams accordance with code's requirements, in which do not need to reinforce diagonal bars. However, one recommendation is that the option 2 in ACI 318-08 can be used for strong seismicity regions only, and specimen CB7 can be designed for low-to-moderate seismicity regions. Moreover, the diagonal bars shall be provided for the coupling beams with $l/h \leq 2$ (Cheng [46]).

(3) Anchored bend of diagonal bars: About bending angle of diagonal bars into the core wall (Figure 2.15), as beam-column joint behavior, they shall be completely bent with reasonable angle and direction. However, they shall be satisfied requirements on anchored bending angle and the development length in codes for tension or compression members, the development length in tension member is recommended. Both EN 1998-1:2004 and ACI 318-08 do not have guidance for this case. Moreover, unfortunately, experimental tests are investigated not to mention the problem for bending angle for diagonal bars in the coupling beams. In case of anchored bend for diagonal bars, critical section is noted to calculate the development length. There is one solution for bending angle of the diagonal bars as shown in Figure 4.7.

(4) Anchorage of horizontal bars: Fully anchored longitudinal bars are required. Figure 4.7 shows state-of-art solution for the coupling beams, this option is combined details in Figure R21.9.7.b (ACI 318-08) with experimental result by Lequesne et al. [47].

(5) About problem of drop of the coupling beams and replace by slab: It is violated EN 1998-1:2004, Clause 5.5.3.5(1)P: "Coupling of walls by means of slabs shall not be taken into account, as it is not effective." More specific requirement likes this provision is needed to add in standards and codes.

Finally, all issues of the coupling beams are very clear according to both codes and experimental tests. However, the width of core wall is problem because maybe it should be designed and detailed larger by requirements from the coupling beams.

CHAPTER V
DEEP BEAMS IN HIGH RISE BUILDINGS

5.1 EN 1998-1:2004

According to EN 1992-1-1:2004, Clause 5.3.1(3) has definition "A beam is a member for which the span is not less than 3 times the overall section depth. Otherwise it should be considered as a deep beam."

Deep beams should normally be provided with an orthogonal reinforcement mesh near each face, with a minimum of reinforcement ratio. The distance between two adjacent bars of the mesh should not exceed the lesser of twice deep beams thickness or 300 mm. Reinforcement, corresponding to the ties considered in the design model, should be fully anchored for equilibrium in the node in the design model, by bending the bars, by using U-hoops or by anchorage devices, unless a sufficient length is available between the node and the end of the beam permitting an anchorage length of l_{bd} (Clause 9.7 in EN 1992-1-1:2004).

Where a non-linear strain distribution (discontinuity regions or D-region) exists (e.g. supports, near concentrated loads or plain stress) strut-and-tie models (STM) may be used (Clause 6.5.1(P) EN 1992-1-1:2004). Strut-and-tie model is given in EN 1992-1-1:2004: Clause 5.6.4-Analysis with struts and tie models, and Clause 6.5-Design with strut and tie models.

5.2 ACI 318-08

The deep beams will be designed and detailed as member subject to shear load, flexural or axial load or to combined flexure and axial loads.

Clause 10.7.1 requires "Deep beams are members loaded on one face and supported on the opposite face so that compression struts can develop between the loads and the supports, and have either:

(a) clear spans, l_n, equal to or less than four times the overall member depth; or

(b) regions with concentrated loads within twice the member depth from the face of the support."

The deep beams shall be designed either taking into account nonlinear distribution of strain, or by strut-and-tie models. Lateral buckling shall be considered.

V_n for deep beams shall not exceed: $10b_w d(f'_c)^{1/2}$.

For detail requirements of the deep beams, such as minimum area of flexural tension reinforcement, minimum horizontal and vertical reinforcement in the side faces of deep beams, are satisfied Clause 10.7.3, 10.7.4, respectively. The anchorage of positive and negative moment tension reinforcement in the deep beams subject to flexural load shall be designed in accordance with Clause 12.10.6 (Adequate anchorage shall be provided for tension reinforcement in flexural members), and 12.11.4 (Anchorage at simple support, interior support), 12.12.4 (Anchorage at interior support). The longitudinal reinforcement in the deep beams should be extended to the supports and adequately anchored by embedment, hooks, or welding to special devices; and bent-up bars are not recommended. And Appendix A-Strut-and-Tie Model (ACI 318-08) shall be applied for other design and detail requirements for the deep beams.

5.3 Discussion

Firstly, as regards to analysis model for the deep beams, the EN 1992-1-1:2004 (not EN 1998-1:2004) and Chapter 10, ACI 318-08 introduce on STM method for determining the shear strength of reinforced concrete deep beams. The main concepts of

STM are struts (compression member), ties (tension member), and nodes (joint member) (Nagarajan et al. [52]). The STM for the deep beams (Figure 5.1) consists of two concrete compressive struts, longitudinal reinforcement serving as a tension tie, and joints referred to as nodes. The concrete around a node is called a nodal zone. The nodal zones transfer the forces from the inclined struts to other struts, to ties and to the reactions (Bower [53]). Nodes are described by the type of the members that intersect at the nodes. For example, a CCT node is one, which is bounded by two struts (C) and one tie (T). Nodes are classified as CCC, CCT, CTT or TTT (Nagarajan et al. [51]). Principle of STM method is equilibrium condition only and STM is applied for design of local regions (Hsu [54]). The STM shall be in equilibrium with the applied loads and the reactions.

For inclined angle between the strut and the tie, ACI 318-08 requires the smallest angle between the strut and the tie in a D-region is arctan(1/2)=26.5 degrees, rounded through 25 degrees, and inclined angle is recommended θ: $25^o \leq \theta \leq 65^o$. While for EN 1992-1-1:2004, $\theta=31^o$ though 59^o. The inclined angle is not allowed less than θ_{min} in preventing too long tie and too short strut. ACI 318-08 does not contain detailed requirements for designing deep beams for flexure except that nonlinearity of strain distribution and lateral buckling is to be considered. Suggestions for the design of deep beams for flexure are given in References 10.22, 10.23, and 10.24. (ACI 318-08, R10.7).

Clause 11.8 (ACI 318-08) applies only to single span deep beams. Continuous deep beams can be applied STM. About reinforcement layout in the deep beams, longitudinal bars should be arranged along the height of the deep beams to support the transverse reinforcement along the deep beam axis, not only arranged at the bottom of the deep beams to subject bending moment as normal beams (Nawy [55], Tuan [56]). In design, shear force in the deep beams is major consideration. The ratio and space of both the vertical and horizontal shear reinforcement differ considerably from those used in the normal beams, as well as the expressions that have to be used for their design (Nawy [55]). According to Kong [57], the deep beams can be designed as normal beams in flexural strength. While for shear strength, can not use formulas of the normal beams to calculate the deep beams, and STM will be applied. According to ACI 318-08, the design principle is based on:

$$V_u \leq \phi V_n \tag{5-1}$$

$$V_n = V_c + V_s \tag{5-2}$$

where: V_u is the design shear force at the critical section;

V_n is the nominal shear strength;

V_c is the shear strength provided by concrete;

V_s is the shear strength provided by steel;

ϕ is the capacity reduction factor for shear, taken as 0.75.

In simply supported deep beams, design of shear strength is carried out for the critical section. But for continuous deep beams, shear strength design are not based on the design shear force at the critical section. Instead, the shear reinforcement at any section is calculated from the design shear force V_u at that section. And for continuous deep beams, the concrete nominal shear strength V_c is to be taken not the same with simply supported deep beams (Kong [57]).

Following the increased interest in STM regarding complex load states in high rise buildings, general methods for the application of STM began to appear. This method provided basic concepts and tools that could be applied to complex structures for designs based on behavior models. STM began appearing in North American codes for general design use. The Canadian CSA A23.312 was the first to adopt STM in 1984.

STM have been applied in the EN code before the ACI code. ACI introduced STM provisions in the 2002 edition (ACI 318-02). The STM provisions of ACI 318-05 were written largely by compiling information and provisions from European codes (Brown and Bayrak [58]). The ACI 318-02 is only for reference and introduction, and ACI 318-08 will be officially applied. The STM method uses the principle of lower bound which gives a conservative result. Simplicity of STM in modeling and analysis makes it a valuable tool that may be used by almost students and structural engineers for design of complex or unusual structural concrete members. The STM also has many difficulties; one of them is that the sketch of trajectories of the principle stress distribution in reinforced concrete components. This should have experience in the process of selecting a model for specific components. For one component may have many different models for calculating and will give different results. The combined with finite element method to determine accurately the trajectories of principle stress distribution and based on model that is selected model to reflect the proper working of the discontinuous regions (Tuan [56], Ley et al. [59]). The greatest difficulty of STM is to build a good model for the considered element or region. One of the most important issues here is the correct selection of inclination of compressive struts (Baczkowsk [51]).

Continuous deep beams occur as transfer girders in reinforced concrete frames in high rise buildings, as pile caps and as foundation wall structures, etc. The definition of continuous deep beams is not formally in EN 1998-1:2004 and ACI 318-08. Frederick and Jonathan [60] shown that ACI 318 defines deep beams as flexural members with clear span-depth ratios less than 2.5 for continuous spans and 1.25 for simple spans. According to Kong [57], the ACI deep beams definition is based on shear behavior while EN definition is based on flexural behavior. When considering the design recommendations in two codes will recognize the different definitions. Continuous deep beams behave differently from either simply supported deep beams or continuous shallow beams. Continuous deep beams develop a distinct 'tied arch' or 'truss' behavior not found in shallow continuous beams. And the result of this is that detailing rules of conventional reinforcement, based on shallow beams or simply span deep beams, are not necessarily appropriate for continuous deep beams. In simple span beams, the region of high shear coincides with a region of low bending moment. In continuous beams, the locations of maximum negative moment and shear coincide, and the point of inflection may be very near the critical section for shear. At an interior support in a continuous beam, the zone of high shear and high negative bending moment coincide. These differences cast further doubt on the usefulness of empirical equations based on simple span experimental test data. The usual design practice for continuous deep beams has been to employ empirical equations, which are invariably based on simple span deep beams experimental data tests.

For design of the deep beams, especially for practicing engineer, it is recommended that the equilibrium truss model to be use, Zhang and Tan [61] presented STM method for two span continuous deep beams using truss model as shown in Figure 5.2.

The experimental tests by Rogowsky et al. [62] on continuous deep beams with column stub showed that the load capacity of continuous deep beams would not be properly estimated by formulas developed for simple deep beams. Therefore, proper design of continuous deep beams would require further investigations to understand the influence of various parameters on their load capacity. Brown and Bayrak [58, 63] checked results of 596 experimental tests of beams with shear span-depth ratios less than two were compiled from the technical literature. Laboratory tests of beams with small shear span-depth ratios are among the simplest types of structural members for which strut-and-tie is appropriate. Typically, laboratory specimens are subjected to one or two concentrated loads and have simple supports. In fact, however, shapes of deep beams or transfer beams are complex, they are not simple as experimental tests, examples and analysis data in papers and text book. Yang and Ashour [64] investigated total of 75 two spans with top loaded reinforced concrete deep beams were compiled from different sources, such as Rogowsky et al [62]-1986, Ashour-1997, Subedi-1998, Asin-1999, and Yang et al.-2007. Figure 5.3 illustrates STM for continuous deep beams on ACI 318 code. All beams were reported to fail in shear due to a major diagonal crack within interior shear spans, joining the edges of load and intermediate support plates. Figure 5.4 showed crack patterns in simple and two span deep beams tested by Rogowsky et al. [62].

Many experimental tests, design examples; results of analysis, checking and evaluation; proposals and recommendations, for a span and two span deep beams, deep beams with opening, under one and two point loadings, have been done by researchers, such as Untrauer and Siess [65], Rogowsky et al. [66], Ove Arup [67], Nawy [55], Rogowsky et al. [62], Hwang and Lee [68], Kong [57], Reineck [69], Aguilar et al. [70], Wight and Parra-Montesinos [71], Nilson et al. [33], Dirar and Morley [72], Singh et al. [73], Ong [74], Zhang and Tan [61, 75], Yang and Ashour [64], and Arabzadeh et al. [76]. Since, it is difficult to practicing structural engineers in the fact. Some design recommendations for ACI 318-05 has introduced by Brown and Bayrak [63] after they checked results of 596 experimental tests. Particularly, experimental tests have been done by Ley et al. [59], Untrauer and Siess [65], etc. with bearing plates at loading point and support locations as shown in Figure 5.5, and other ones have been carried out by Rogowsky et al. [62, 66] with column stubs as shown in Figure 5.6, column stubs are more clearly similar to practice model than bearing plates. Reineck [69] collected and presented many examples for the structural concrete design with STM according to ACI 318 code, in which bearing plates are provided at all loading and support locations as shown in Figure 5.5.

There are some the methods and its discussions for deep beams analysis, such as elastic analysis, finite element analysis, ACI 318 (1977), Kong, Robins and Sharp (1975), truss models (Kong 2002), nonlinear finite element analysis (Dirar and Morley [70]), Strut-and-tie design methodology for three-dimensional reinforced concrete structures (Leu et al. [77]), 2D finite element analysis (Ong [74]), micro truss model (Nagarajan et al. [52]). Quangfeng and Hoogenboom [78] also have been presented other method, namely “stringer-panel model”. Method of “stringer-panel model” is intermediate method between strut-and-tie models and finite element method (Tuan [56]). There are some other methods based on STM: Softened strut-and-tie models of Hwang and Lee [68], modified strut-and-tie models of Zhang and Tan [61], simple strut-and-tie models of Arabzadeh et al. [76]. According to Arabzadeh et al. [76], some existing methods are simplified softened truss model of Mau Su (1989), combined softened STM of Matamoros et al. (1986), formula proposed by Foster-Gilbert based on plastic truss model (1996). There are three methods for formulating strut-and-tie models: Elastic analysis based on stress trajectories, load path approach, experimentally test. Table 5.3 shows existing problems for the deep beams in practice.

Basic design procedure for a structures using STM according to ACI 318-08 Appendix A is as following below:

(i) Definition of structural system, determination of loads and reactions, estimate dimensions and sizes of members.

(ii) Definition of B and D-regions.

(iii) Design for B-regions as normal beams in flexural strength (by other methods).

(iv) Design for D-regions: Design of struts, ties, and nodal zones shall be based on:

$$\phi F_n \geq F_u \tag{5-3}$$

(a) For struts:

- The nominal compressive strength of a strut without longitudinal reinforcement:

$$F_{ns} = f_{ce} A_{cs} \tag{5-4}$$

The effective compressive strength of the concrete in a strut:

$$f_{ce} = 0.85 \beta_s f'_c \tag{5-5}$$

- The nominal strength of a longitudinally reinforced strut:

$$F_{ns} = f_{ce} A_{cs} + A'_s f'_s \tag{5-6}$$

(b) For ties:

- The nominal strength of a tie shall be taken as

$$F_{nt} = A_{ts} f_y + A_{tp}(f_{se} + \Delta f_p) \tag{5-7}$$

where $(f_{se} + \Delta_{fp})$ shall not exceed f_{py}, and A_{tp} is zero for nonprestressed members.

(c) For nodal zones:

- The nominal compression strength of a nodal zone shall be

$$F_{nn} = f_{ce} A_{nz} \tag{5-8}$$

The effective compressive strength of the concrete in the nodal zone:

$$f_{ce} = 0.85 \beta_n f'_c \tag{5-9}$$

(v) Arrangement and detail of reinforcements.

Appendixes A shows calculation for examples of simple deep beams using data tested by Rogowsky et al. [62] according to ACI 318-08, Appendix A. Model of specimens was similar to practice shape about column stubs. Results are quite conservative, it may be due to data of experimental test had been done for long time ago.

Almost experimental tests are listed above subjected to monotonic loadings. Since, current provisions of STM are based on theoretical and/or experimental tests that depart from assuming monotonic loadings. Structural members, however, are often subjected to cyclic demands, such as earthquake loading. There is little evidence on the behaviour of members designed using STM and subjected to reversed cyclic loads. The doubts therefore have been cast upon STM being appropriate for seismic design. The adequacy of using STM for seismic design was assessed by Alcocer and Uribe [79]. In four tests, due to loading equipment availability, upward loading (negative direction) was limited to approximately half of the maximum load that could be applied in the positive direction. Figure 5.7 demonstrate final crack patterns for all deep beams, first two beams (MT, MR) were tested under monotonic loads, and last two beams (CT, CR) were tested under reversed cyclic loads. Hysteretic loops of beam CT and CR are shown in Figure 5.8. Hysteretic loops show considerable pinching, especially at deflections of 20 mm, and severe stiffness degradation. The hysteretic loops observed are typical of structural members failing in shear. Average rotations at beam strength under positive loading were 2.3% and 1.6% for beams CT and CR, respectively. First cracking, first yielding, and last yielding recorded before failure are also indicated in Figure 5.9. The curves show a nearly elastic behavior up to a deflection of approximately 9 mm. At this deflection, the first yielding occurred in the transverse reinforcement (beams MT, CT, and CR) or in the longitudinal reinforcement (beam MR). In terms of strength, stiffness, and deformation capacity, the performance of all four beams tested exceeded expectations, thus verifying the reliability of STM for structural members subjected to either monotonic or reversed cyclic loads. The STM may be used for seismic design of structural members subjected to reversed cyclic shear demands up to $0.42\sqrt{f'_c}$ (MPa) ($5.0\sqrt{f'_c}$ [psi]) and to inelastic deformation demands up to 2.3%. For these cyclic deformations and shear demands, code provisions on strut and node strengths need not be altered. This conclusion departs from an assumed failure mode controlled by yielding of the tie reinforcement. The predicted strengths applying the provisions of Appendix A of ACI 318-05 were smaller than the actual strengths obtained in the laboratory by an average of 27%. One of the most important from this experimental test is that current provisions, ACI 318-05, for STM may be used in seismic design.

Regardless of analysis problems of transfer structures as transfer beams, according to Puvvala [80], unlike normal deep beams, there is no particular span to depth ratio for estimating structural behavior and failure mechanism of transfer beams. Transfer beams behave either as full tension, deep beams or as normal beam in flexural moment depending on type of upper structure form as well as relevant parameters such as span to depth ratio of transfer beams, stiffness of support columns, span of shear wall and degree of coupling on the coupled shear wall… There are 3 major problems are introduced by Puvvala [80]: Single-span and two-span continuous transfer beams supporting in-plane loaded shear walls (Figure 5.10, 5.11), transfer beams supporting in-plane loaded equal and unequal coupled shear walls (Figure 5.12), and frame structures supported on transfer beams (Figure 5.13). Method of analysis, modeling by finite element method, structural behavior, relevant parameters, and stiffness of the coupling beams in case of coupled shear walls also presented. Transfer beam should be designed as flexural-tension members, but not ordinary beams in bending or normal deep beams. Li [81] used non-linear finite element analysis to investigate failure modes, failure loads and load transfer mechanism, and introduced some formulas for problem of transfer beams support an in-plane loaded shear wall. This mode of failure changes from shear failure to flexural-shear failure then turning into flexural failure according to different span to depth ratio or width of beam. When the stiffness of column is enough, the transfer beams likes fixed ends beam. Otherwise, it likes simple supporter beam. With proposed formulas, flexural moment and axial force occurred at mid-span of transfer beams may be calculated simply, quickly, effectively. Table 5.4 and Figure 5.14 indicated that, with the width of beam is fixed at 2 times of the width of shear wall, and span to depth ratio of transfer beams range from 2 to 12, cracking load decrease slowly with increase in the span to depth ratio, so that change of ratio for span-depth has little effect to cracking loads. Figure 5.15 demonstrated that failure load decreases very little when span to depth ratio change from 2 to 3. Nevertheless, failure load then decreases quickly with span to depth ratio from 3 to 12. It means that the depth of transfer beams is larger than 1/3 of total span, it may not be useful to increase the depth for getting larger failure load. Regardless of changes for the width of transfer beams, with span to depth ratio of the beam fixed to 6, the width of transfer beams ranges from 0.5 to 5 times of the width of upper shear wall, Table 5.5 and Figure 5.16 showed that cracking load increase nearly along a straight line with increase in width ratio. Since, the change of the width ratio has greater effect on cracking load than that of the change of span to depth ratio. For the failure load in Figure 5.17, failure load only increase rapidly when the width ratio from 0.5 to 3. It indicates that the effect of the width beam is great as the width ratio is less than 3. The span to depth ratio of the transfer beams could be ranged from 3 to 8, the width of beam could be chosen from 2 to 3 times of the width of the shear wall. According to Kuang and Li [82], method of box foundation analogy could be applied to design of the transfer beams (Figure 5.18). A comparison of results between the proposed formulas based on box foundation analogy and the finite element method as shown in Table 5.6, it is clear to see all results are quite similar.

Continuously, about transfer structures as transfer plates, Zhang [83] had been reported on transfer plate-shear wall systems, including cases of transfer plates supporting in-plane loaded equal and unequal coupled shear walls by considering the interaction between the transfer plates and upper structures as shear walls. Internal forces and stresses in transfer plate will reduce after taking the interaction into consideration. Other parameters of problem for transfer plates are similar to transfer beams as mentioned above. Two steps are usually needed to complete design it: (i) Model the transfer plate into an equivalent grillage system with other members as beam and slab, and then analysis whole structure to get internal forces, deformations, stresses of all structures; (ii) Because of its great important, transfer plate needs to be analyzed carefully and thoroughly by using finite element method with its loads which are the same as internal forces of the adjacent members at the interface between the transfer plate and upper structures. Zhang [84] presented for the development of a new flat shell element that is tailored to the analysis of transfer plates that connecting to frame structures and shear walls/core walls in tall buildings. This new element is eight-node flat shell element with six degrees of freedom at each node. There are three method of analysis of transfer plates (Zhang [84]): (i) Rigid base analogy, in which whole structures can be divided into two parts along the transfer plate. The first part includes the transfer plate and the structures above the transfer plate. The transfer plate is assumed to be a rigid base. The first part is analyzed by finite element method, then its internal forces are applied to the lower part as the external loads. The most disadvantageous limitation of this method is that structure systems has to be divided into two parts and each part is modeled separately. This results in loss of the interaction between transfer plate and other parts of building; (ii) Grillage system method, with beam element are employed to set up a grillage system illustrated to replace the transfer plate. This grillage system is able to connect with the members of other parts of structure system. Therefore, all structures can be modeled in one model. Some limitations of this method are: It is not easy to set up a grillage system that has identical flexural stiffness with the original transfer plate and not possible to calculate the resulting internal forces in the transfer plate from the grillage system if internal forces of each grillage element have been obtained; (iii) Flat shell modeling, is to employ the finite element method combining with the membrane element and the plate element. This combination ensures the capacity of modeling the in-plane and bending stiffness. Flat shell method is the most suitable for analysis of transfer plates now.

Li et al. [85] carried out micro-concrete testing model for the non-seismic design high-rise building with 34 stories, was constructed in 1:20 scale, with transfer plate is a reinforced concrete thick plate of 2.7m (Figure 5.21). The model of 1:20 scale was designed according to similitude law. Shaking table tests were conducted and the model was subjected to earthquake loading in level of minor, moderate, major, and supermajor earthquakes for a region of moderate seismicity as given in Table 5.7, with basic seismic intensity at the class VII pursuant to GB 50011-2001. Seismic performance was qualitatively assessed, and it is predicted that the prototype building will not collapse when subjected to major earthquakes, the majority of the damage and failure occurred at the story above the transfer plate (Figure 5.22, Table 5.8). Story drift relates well with the degree of structural damage. For a structural system comprising shear walls, slightly damaged, moderately damaged, and severely damaged would occur if the story drift approaches 1/1,000, in the range of 1/300-1/700, and in the range of 1/80-1/200, respectively (Table 5.9). However, it is considered carefully about difference on material between testing model and real structure, it may be caused incorrectly results. In this test, it is assumed that the micro-concrete can provide reasonable representation of the inelastic behavior of the concrete in the prototype building (Li et al. [85]). A one more specimen tested by Li et al [86], Figure 5.23 demonstrated 18-story building with a transfer plate designed with no seismic provisions. A test specimen in 1:4 scale was used to represent the first two stories as shown in Figure 5.24. For this specimen, model's material was similar to real structure's material. The Pseudo-dynamic tests with substructure technique were conducted using three types of time-history records. The shear walls remained elastic throughout the loading histories, whereas the transfer plate was severely damaged when subjected to an El-Centro Earthquake with a maximum acceleration at 0.64g. Major damages occurred at the transfer plate (Table 5.10, 5.11). The transfer plate may have sufficient strength to resist possible earthquake loading that could be expected in region of moderate seismicity, i.e., 0.16g of an El-Centro Earthquake. However, there is insufficient seismic resistance if maximum acceleration of an El-Centro Earthquake is greater than 0.32g (Table 5.12). Figure 5.25 shows the increase in the vertical displacements at the center of the transfer plate when subjected to the El-Centro Earthquakes with maximum accelerations at 32%g and 64%g. At 0.64g, the vertical displacement of the transfer plate did not return to the initial configuration when unloaded. There was substantial structural damage at the transfer plate. The displacement of the transfer plate at the center of wall is approximately 0.4 mm, leading to a maximum displacement at 2.59 mm. Limitations of drift ratio are 0.5%, 0.5%, and 2.0% according to EN 1998-1:2004, 4.4.3.2, ACI 318-08, 21.13.6, and UBC 1997 Volume 2, 1630.10.2, respectively.

In China, many researches have been carried out for transfer plates and a numbers of tall buildings with transfer plates have been built. However, its design is based on engineering experience. Since, practical stress is much less than design strength, which indicates that the current design method may be too conservative. Consequently, unreasonable reinforcement in transfer plates may cause some negative effect besides a lot of waste of material and human resources (Zhang [83]).

Regardless of continuous transfer beams in building mentioned in Chapter 2, designer group did analysis all shear walls and transfer beams simultaneously. Kuang and Puvvala [87], Viswanath et al. [88] present an analysis method for coupled shear walls supported on continuous transfer beams framing into columns. In this method, the structure system will be divided into two parts. The coupled shear wall is analysis first with assuming ideal fixed support. Then walls internal forces at the level of the transfer beams are defined as external forces for analyzing of the transfer beams. Some charts used in design process. Finite element simulation was used to capture and interpret the interaction mechanism of the system. Effects on structural behavior of system are also presented: Different wall and support system features, interaction between shear wall and transfer beams in considering interior and exterior column interaction effects; and some relevant parameters that significantly influence the force transfer mechanism and structural behaviour, for instance, span-depth ratio of the transfer beams, the stiffness of the support columns.

Secondly, in application of the deep beams or transfer structures in seismic regions, transfer structures can be defined as either flexural or shear structures which transmit heavy loads from columns or walls acting on its top and redistribute them to the supporting columns or walls. Various forms of transfer structures are presented in high and low-rise buildings, such as transfer plates or transfer girders employed for high-rise residential and commercial buildings, whilst transfer beams are commonly used for low or medium-rise buildings (Li et al. [89]). Before 2006, the concept of deep beams and STM did not exist in Vietnam's standards (Table 2.1). In Vietnam standard, TCXD 198:1997 [7] High rise building-Guide for Design of Monolithic Reinforced Concrete Structures, for high rise buildings with 25 stories and total height is not great than 75 meter (Clause 1), Clause 3.3.1 illustrates solution should not to use (Figure 2.17.a), and measure to overcome it as shown in Figure 2.17.b with option as transfer structures or transfer plates, which typically have soft story problem.

Both ACI 318-08 and EN 1998-1:2004 are not considered the deep beams as structures resist against earthquake loading. The provisions for the deep beams are not included in EN 1998-1:2004 as well as ACI 318-08 Chapter 21-Earthquake-resistant structures (Table 5.1). Or without additional provisions in case of design against earthquakes, while the other members in both codes have additional regulations in case design and detail for earthquake resistance. May be due to deep beams are very large stiffness, low ductility, deep beam's plastic hinge mechanism does not form at beam first, so that not benefit for structures resist earthquake loading according to principle of strong column weak beam.

Design of the deep beams for seismic force resisting system are violated seriously following provisions: Clause 4.4.2.3(3)P, Clause 4.4.2.3(4) in EN 1998-1:2004, and Clause 21.6.2.2, Clause 21.6.2.3 in ACI 318-08 (Table 5.2):

- Clause 4.4.2.3(3)P: In multi-storey buildings, formation of a soft storey plastic mechanism shall be prevented, as such a mechanism might entail excessive local ductility demands in the columns of the soft storey.

- Clause 4.4.2.3(4): In frame buildings, the following condition should be satisfied at all joints of primary or secondary seismic beams with primary seismic columns: $\sum M_{Rc} \geq 1.3 \sum M_{Rb}$.

- Clause 21.6.2.2: The flexural strengths of the columns shall satisfy $\sum M_{nc} \geq (6/5) \sum M_{nb}$.

- Clause 21.6.2.3: If Clause 21.6.2.2 is not satisfied at a joint, the lateral strength and stiffness of the columns framing into that joint shall be ignored when determining the calculated strength and stiffness of the structure. These columns shall conform to 21.13.

Clause 21.13 is provisions for members not designated as part of the seismic-force-resisting system. Requirements of 21.13 apply to frame members not designated as part of the seismic-force-resisting system in structures assigned to SDC D, E, and F. Table 2.3 also shows clearly that design of the deep beams as transfer structures, transfer beams shall be prohibited in strong seismic regions with $a_g \geq 0.08g$ in Vietnam. In order to use transfer structures to satisfy current seismic standard, parts under transfer structure include transfer structures shall be designed in elastic behaviour (very rigid), upper parts shall be designed in hinge plastic formation (strong column and weak beam).

However, it is considered about Hong Kong and some Southeast Asia countries, like other mega-cities including Singapore, Bangkok, Shanghai, London and New York, are not located at the high seismicity regions as above-mentioned studies (Table 2.4 and Figure 2.22, 2.23, 2.24). Existing buildings with transfer structures in Hong Kong, following the traditional design codes, British standard (BS), have not been designed for seismic resistance (Li et al. [89]). The highest seismic zone of China, seismic intensity IX with PGA≥0.4g, is equivalent to UBC-1997 zone 4 with Z=0.3g (Table 2.4, Figure 2.22, 2.23). In P.R.China, the use of transfer structures is allowed only in low-to-moderate seismic zones (maximum seismic intensity of VII corresponding PGA=0.1g) (Su [20]), whilst according to Tsang [21], level of seismic hazard specified by the Chinese code (GB 18306-2001) for Hong Kong is VII-VIII corresponding PGA=0.15g (Table 2.4). The China Academy of Building Research Institute also presented detailed design and construction practices for transfer structures in high-rise buildings in Hong Kong and the neighboring cities of Shenzhen, Guangzhou, along with Beijing capital. More than 20 transfer structure buildings with different forms such as plates, beams, boxes and trusses were reviewed (Li et al. [89]).

Hong Kong is located in a low to moderate seismicity zones, peak ground earthquake acceleration ranges from 0.1g through 0.15g only over a 475-year return period (according to GB18306-2001) for structures with typical functionality on firm ground, is well within the typical limit of 0.05g to 0.25g for low-to-moderate zones (Su [20]). On the contrary, EN 1998-1:2004 and TCXDVN 375:2006-Part 1 has the same criteria for seismicity classification. Clause 3.2.1(4) (TCXDVN 375:2006-Part 1) and Clause 3.2.1(4) (EN 1998-1:2004) require that $a_g \geq 0.08g$ is used for strong seismicity regions, with the same 475-year return period. Maximum value of peak ground acceleration of Hong Kong is about double but it is located low to moderate seismicity regions (Table 2.4). Reasons for this issue is that before Hong Kong handed over by People Republic of China, Hong Kong was only very small zone and it adopted English standard (BS) as its code, and now Hong Kong likes province in China and using China’s codes. In the past ten years (from 2006), over 70% of the residential buildings in Hong Kong were constructed using transfer structures (Li et al. [85]). In Vietnam, after TCXDVN 375:2006-Part 1 adopted, some existing projects in Ho Chi Minh city and others provinces are faced problems with transfer structures, because these cities are located in strong seismic regions, while before that, they were located in very low and low seismic regions (Table 5.13). Li [81] also presented 40 practical design examples of transfer beams in P.R.China, including Hong Kong. Modern seismic codes are difference from previous codes by living condition may be higher more and more, or what else?

Regardless of transfer structures in Hong Kong, according to Ho [90], investigation on the ductility demands on reinforced concrete moment-resisting frames for regions of moderate seismicity, based on two existing non-seismically designed and detailed reinforced-concrete frame buildings. The seismic responses of these as-built structures are subjected to an unexpected low-to-moderate seismic shaking. It is theoretically established that the demand of displacement ductility of non-seismically designed and detailed framed building for low-to-moderate earthquakes could vary between 2.0 and 4.1. In addition, the experimental investigations on large-scale frame and shear wall-frame specimens, which without seismic design and detailing, values of the displacement ductility of the frame and the shear wall-frame are similar and may not exceed 3.84 and 3.73. And many recent studies have indicated that the inherent ductility may vary between 1.5 and 4.0. The experimental values match with the value of 3 as recommended for reinforced concrete structures with limited ductility. It has been concluded that the non-seismically designed reinforced concrete structures in Hong Kong may be very marginal or may not be adequate to resist low-to-moderate seismic events. For checking this ductility, future works are needed to calculate inelastic problems (push over) for some non-seismically designed and detailed typical real structures in Vietnam, Hong Kong, and then compare values of computed ductility according to current codes. This is also recommendation to ensure safe factor for proposal on changing for current Vietnamese standard TCXDVN 375:2006 Part 1 as mentioned.

5.4 Brief summary

(1) For analytic model for complex deep beams: For design of the deep beams and continuous deep beams, the STM is a unified approach that considers all load effects simultaneously. The STM approach evolves as one of the most powerful methods for design of shear critical structures and for other disturbed regions in concrete structures. The model provides a rational approach by representing a complicated structural member with an appropriate simplified truss models. But there are no single and unique STM for most design situations. For design of continuous deep beams in high rise buildings, continuous deep beams can be treated in the same manner as simply supported deep beams, except that additional reinforcement has to be provided for the negative moment at the support, at the continuous supports, the total sections is in tension (Nawy [53]). There are some the methods available for analyzing deep beams, but STM, softened strut-and-tie model (SSTM), simple truss model, 2D and 3D finite element method, are recommended to apply.

The design and detail for deep beams have been provided in the ACI 318-08 and and EN 1992-1-1:2004, but in some cases, its application in practice is still limited by the complexity of the structural system in high rise buildings. In some complex cases, STM can not completely fit to calculate the deep beams, by the complex deep beams is usually different with normal deep beams in that deep beam width is so much larger due to support many complex structures on it. Many experimental tests and research have been introduced but only for the simple deep beams, such as the simply supported deep beams, one or two spans, the width of deep beams is in normal cases. In order to calculate deep beams and transfer beams in high rise buildings, the both codes are applied STM. Only solution is simplification for structural diagram to have simple deep beams. Of course, the best solution is to use no deep beams in structural system or if need to design deep beams, simple forms for deep beams, and deep beams which have been studied and calculated are encouraged. The deep beam type of transfer structures is the simplest one among all transfer structures, such as transfer beams, transfer plates, transfer trusses, transfer boxes…

Finally, to continue to need more research for deep beams and transfer beams with different forms. Since deep beams are often designed in high rise buildings built under seismic conditions, both ACI 318-08 and EN 1998-1:2004 should be considered to add the specific terms for deep beams. However, but while there are no more accurate methods or methods of more simplicity and safety, for construction projects in strong seismic regions, should have clear provisions to restrict or prohibit to use this solution, this will be prevented a solution from far and so will be better for the projects constructed in strong seismic regions.

Analysis problems of transfer structures as transfer beams and transfer plates: Transfer beams and transfer plates can be divided into transfer beam/plate-shear wall systems or transfer beam/plate-column systems. Design of transfer beams, transfer plates (including dimension of members, thickness, reinforcement layout, etc), are still mainly based on finite element method, engineering practice and experience. Using flat shell element to model transfer plates is still the most suitable method for analysis of transfer plates.

Both EN 1998-1:2004 and ACI 318-08 were not allowed to use transfer structures in strong seismicity regions. However, transfer structures were not mentioned in lower seismicity regions. Since, Vietnamese seismic code shall be re-considered by transfer structures should be applied in Vietnam.

(2) Application of the transfer structures in seismic regions: In design of the deep beams, transfer structures for seismic force resisting system, provisions of Clause

4.4.2.3(3)P, Clause 4.4.2.3(4) in EN 1998-1:2004, and Clause 21.6.2.2, Clause 21.6.2.3 in ACI 318-08, are violated seriously. It means that another lateral load resisting elements shall be provided to replace for lateral earthquake load resisting system with transfer beams.

In Vietnam, according to TCXDVN 375:2006-Part 1 (also is EN 1998-1:2004), the deep beams as transfer beams, transfer structures shall be prohibited in strong seismicity regions where $a_g \geq 0.08g$. Otherwise, general speaking, transfer structures and transfer beams can be used but they shall be designed for buildings in very low to low seismicity regions. Unfortunately, by reasons of design standards for earthquakes resistance and criteria of earthquake intensity classification, some construction projects with transfer structures as transfer beams were built before 2006 are now unsafe under current seismic standards.

Transfer structures are widely used in Southeast Asia, like Thailand, Singapore, Malaysia, especially Hong Kong, where located in low-to-moderate seismicity regions. Obviously, the problem for transfer structures now is similar to wide beam-column connections in Vietnam as mentioned in Chapter 3. As the same the wide beam-column connections, the author recommend that it is considered to modify limitation of peak ground acceleration for low and strong seismicity regions in TCXDVN 375:2006-Part 1 as shown in Table 2.5, 2.6 and Table 2.7, with $0.05g \leq a_g < 0.20g$ for low or low-to-moderate seismicity regions. It leads to use transfer structures in many regions in Vietnam to provide flexibility in different architectural arrangements above and below the transfer structures. Vietnamese seismic standard also should be considered by values of peak ground acceleration, a_g, are taken from notes in Clause 3.2.1(4) and 3.2.1(5)P, EN 1998-1:2004, these notes are only recommendations by EN 1998-1:2004.

CHAPTER VI

CONCLUSIONS

This thesis aims to study the performance of three typical problems which the author faced during working in Vietnam:

- Wide beam-column joints and beam-core wall joints.

- Coupling beams.

- Deep beams in high rise buildings.

For the first problem, both ACI 318-08 and EN 1998-1:2004 do not have provisions relevant to this one. EN 1998-1:2004 has provisions for calculation of wide beam-column joints with condition of column size is less than the wide beam width, while ACI 318-08 has no any recommendations. Both codes have been mentioned about case of the wide beam with provisions for transverse reinforcement through the column core to confine beam longitudinal reinforcements passing outside the column core, but EN 1998-1:2004 has not clear provisions about this transverse reinforcement. The development length is needed to anchorage into joint for beam longitudinal bars outside the column core are not required in both codes. For beam-core wall joints, generally, it may be considered as conventional beam-column joints as well as wide beam-column joints in some specific cases. Otherwise, joints are designed and detailed in accordance with code’s requirements to ensure safety. Especially, the core wall section should be expanded to enough for longitudinal reinforcement of the beam and wide beam that anchorages at joints, and needed to control strength and detailing of connections.

One of solution to decrease reinforcement ratio in joints is changed from rigid to hinge connection. The solution is available for only local minor joints in structural system, it means that for links between non-structural members or second seismic members. Otherwise, stability condition of whole structures shall be affected deeply, the core wall will be became wasted and redundancy member by not receive lateral loading transferred through beams due to hinge connection. Generally, in case of the wide beam, eccentric wide beam-column joints and beam-core wall joints should be restricted or prohibited to design for earthquake-resistant structures, especially in high rise buildings and in strong seismicity regions, but it is recommended that all code's provisions needed to indicate clearly to apply them in practice working. About design criteria of structures for earthquake resistance in strong seismicity regions, according to EN 1998-1:2004-Part 1, Clause 5.2.3.5(1)P requires on structural redundancy: A high degree of redundancy accompanied by redistribution capacity shall be sought, enabling a more widely spread energy dissipation and an increased total dissipated energy. The necessary redistribution capacity shall be achieved through the local ductility rules in standard.

Secondly, diagonal reinforced concrete coupling beams is also complex problem because it is not easy to construct them in site by steel congestion in the coupling beams and core wall, shear wall. Both ACI 381-08 and EN 1998-1:2004 has provisions about the coupling beams, but it is needed to have provision relevant to bending angle for diagonal bars anchored into wall piers. All research has been indicated the important role of diagonal bars in the coupling beams, and many in among research has been also carried out with difference type of the coupling beams. In time without new provisions to construct coupling beams easily, one reasonable solution well be expanded the wall width to fit place for all types of reinforced steels. In thesis, there are also proposals for the coupling beams in low-to-moderate and strong seismicity regions.

Lastly, using deep beams as transfer beams is acceptable option in designing high rise buildings, but tries to use deep beams with simple configuration or must has full data from experimental tests. EN 1998-1:2004 for earthquake resistant structures has no provisions for deep beams, otherwise, EN 1992-1-1:2004 for non-earthquake resistant structures has provisions for deep beams, and the problem is similar to ACI 318-08. Both codes has no any additional provisions for deep beams in design of earthquake-resistant structures, it is different when compare with other members such as beam, column, coupling beams, wall. Transfer structures are also complex members in design of tall building. In some cases, such as residential, commercial and office buildings in strong earthquake regions, transfer structures shall be prohibited completely, such as Hanoi capital and other regions in Vietnam with $a_g \geq 0.08g$ (strong seismicity regions according to TCXDVN 375:2006-Part 1 [6]). For other earthquake zones (very low to low seismicity regions [6]), transfer structures may be applied for low to medium rise buildings, but with some compulsory requirements, such as it must be used typical transfer structures, they had been designed in previous projects, and some other provisions related to contractors, material suppliers, etc. The transfer structures also can be adopted in Vietnam if acceptance of change a_g according to above proposed recommendations.

In summary, in order to narrow gaps between theory and practice, many studies shall be carried out in the future and continue to update in new editions of codes. Some cases are under research process, other ones have not enough data to apply. Since, there are still some practical cases are difference with codes or have no requirement in codes. It is better to design and detail all structures in accordance with code's requirements, except based on reliable experimental studies.

Finally, Vietnam must have official code set with first criteria are consistent and conformity with country's conditions and integrate with common codes. In TCXDVN 375:2006-Part 1, classification on earthquake regions based on design peak ground acceleration, a_g, with $a_g \geq 0.08g$ corresponding strong seismicity regions, this provision may be too safe. The author would like to have recommendation for changing of a_g, using $0.05g \leq a_g < 0.2g$ and $a_g \geq 0.2g$ for low or low-to-moderate and strong seismicity regions to replace $0.04g \leq a_g < 0.08g$ and $a_g \geq 0.08g$ for low and strong seismicity regions, respectively, in Vietnamese current seismic code, TCXDVN 375:2006-Part 1 (Table 2.7), or alternative solution is more conservative with $0.05g \leq a_g < 0.15g$ and $a_g \geq 0.15g$ for low-to-moderate and strong seismicity regions, respectively. If proposed recommendations are approved then problems for wide beam column joints, coupling beams, transfer structures in high rise buildings, will be designed more accurately corresponding to each seismic zones, especially be met real conditions not only on earthquake intensity, but also on construction technology, profits therefore are made by these changes./.

NOTATIONS

a_1	=	effective bearing length
a_g	=	design ground acceleration
a_{gR}	=	reference peak ground acceleration
A_{cs}	=	cross-sectional area at one end of a strut in a strut-and-tie model, taken perpendicular to the axis of the strut
A_{cw}	=	area of concrete section of an individual pier, horizontal wall segment, or coupling beam resisting shear
A_j	=	the effective cross-sectional area within a joint computed from joint depth times effective joint width
A_{nz}	=	area of a face of a nodal zone or a section through a nodal zone
A_{tp}	=	area of prestressing steel in a tie
A_{ts}	=	area of nonprestressed reinforcement in a tie
A_s	=	cross sectional area of reinforcement or area of nonprestressed longitudinal tension reinforcement
A'_s	=	area of compression reinforcement
A_{vd}	=	total area of reinforcement in each group of diagonal bars in a diagonally reinforced coupling beam
A_{s1}	=	area of the beam top reinforcement
A_{s2}	=	area of the beam bottom reinforcement
A_{si}	=	total area of steel bars in each diagonal direction
b_b	=	width of the web of a wide beam
b_c	=	cross-sectional dimension of column

b_e = slab effective width

b_f = width of flange in the boundary element of a wall

b_j = effective joint width

b_o = width of confined core in a column or in the boundary element of a wall (to

centreline of hoops)

b_w = thickness of confined parts of a wall section, or width of the web of a beam

b_{wo} = width of web in the boundary element of a wall

c = factor depend on the roughness of the interface

d = effective depth of section, in Table 3.2 d is diameter of reinforcement

d_b = nominal diameter of bar

d_{bL} = nominal diameter of longitudinal bar

e_o = eccentricity of axial force to the center of cross section

h = overall thickness or height of member or cross-sectional depth

h_b = height of a wide beam

h_{jc} = distance between extreme layers of column reinforcement

h_s = clear storey height or transverse beam depth

h_w = height of wall or cross-sectional depth of beam

f_{bd} = design value of the ultimate bond stress for ribbed bars

f_{cd} = design value of concrete compressive strength

f_{ce} = effective compressive strength of the concrete in a strut or a nodal zone

f_{ck} = characteristic compressive cylinder strength of concrete at 28 days

f_{ctd} = design value of concrete tensile strength

f_{ctm} = mean value of tensile strength of concrete

f_{py} = specified yield strength of prestressing steel

f_{se} = effective stress in prestressing steel (after allowance for all prestress losses)

f_y = specified yield strength of reinforcement

f_{yd} = design value of yield strength of steel

f'_c = specified compressive strength of concrete

f'_s = stress in compression reinforcement under factored loads

F_n = nominal strength of a strut, tie, or nodal zone

F_{nn} = nominal strength at face of a nodal zone

F_{nt} = nominal strength of a tie

F_{ns} = nominal strength of a strut

F_u = factored force acting in a strut, tie, bearing area, or nodal zone in a strut-and-tie model

g = acceleration of gravity

l_{an} = development length

l_{bd} = design anchorage length of longitudinal reinforcement

$l_{b,min}$ = minimum anchorage length

$l_{b,rqd}$ = basic required anchorage length

l_c = length measured from the extreme compression fibre of the wall up to the point where unconfined concrete may spall due to large compressive strains

l_{dh} = development length in tension of deformed bar or deformed wire with a standard hook, measured from critical section to outside end of hook (straight embedment length between critical section and start of hook [point of tangency] plus inside radius of bend and one bar diameter)

l_n = length of clear span measured face-to-face of supports

l_w = length of cross-section of wall

M_{Ed} = design bending moment from the analysis for the seismic design situation

ΣM_{Rb} = sum of design values of moments of resistance of the beams framing into a joint in the direction of interest

ΣM_{Rc} = sum of design values of the moments of resistance of the columns framing into a joint in the direction of interest

p_h = the perimeter of centerline of the beam outermost closed transverse torsion reinforcement

q = behavior factor

R_s = design tensile strength of reinforcement

R_b = design compressive strength of concrete

V_{Ed} = design shear force from the analysis for the seismic design situation

V_{jhd} = horizontal shear force acting on the concrete core of the joints

V_n = nominal shear strength

V_u = factored shear force at section or design shear force at the critical section

V_c = shear strength provided by concrete

V_C = shear force in the column above the joint, from the analysis in the seismic design situation

V_s = shear strength provided by steel

$\Delta\lambda_{an}$ = factor for calculation of development length, given in table

Δf_p = increase in stress in prestressing steel due to factored loads

α = angle between diagonal bars and axis of a coupling beam

α_1 = effect of the form of the bars assuming adequate cover

α_2 = effect of concrete minimum cover

α_3 = effect of confinement by transverse reinforcement

α_4 = influence of one or more welded transverse bars (φ_t>0,6φ) along the design anchorage length l_{bd}

α_5 = effect of the pressure transverse to the plane of splitting along the design anchorage length

β_n = factor to account for the effect of the anchorage of ties on the effective compressive strength of a nodal zone

β_s = factor to account for the effect of cracking and confining reinforcement on the effective compressive strength of the concrete in a strut

δ_u = displacement of the post-ultimate strength curvature at 85% of the capacity of resistance.

δ_{yo} = displacement at yielding.

ε_u = maximum usable strain at extreme concrete compression fiber

ϕ = diameter of a reinforcing or capacity reduction factor for shear

φ_t = diameter of transverse bar

γ = values for Type 1, Type 2 connections, and depends on the connection classification, given in table

γ_I = important factor for buildings

γ_{Rd} = factor to account for overstrength due to steel strain-hardening

η = degree of connection

η_1 = coefficient related to the quality of the bond condition and the position of the bar during concreting

η_2 = related to the bar diameter

λ = modification factor reflecting the reduced mechanical properties of lightweight concrete, all relative to normalweight concrete of the same compressive strength

λ_{an} = factor for calculation of development length, given in table

μ = factor depend on the roughness of the interface, or displacement ductility ratio, $\mu=\delta_u/\delta_{yo}$

ν_d = normalized axial force in the column above the joint

θ = inclined angle between strut and tie in strut-and-tie models

θ_{min} = minimum inclined angle between strut and tie in strut-and-tie models

ρ' = compression steel ratio in beams

ρ_{max} = maximum allowed tension steel ratio in the critical region of primary seismic beams

σ_{sd} = design stress of the bar at the position from where the anchorage is measured from

ω_{an} = factor for calculation of development length, given in table

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Table 1.1-Short history overview of the creation of the Eurocodes [3]

Time	History overview
1975	The Commission of the European Community decided on an action program in the field of construction based on article 95 of the Treaty. The objective of the program was the elimination of technical obstacles to trade and the harmonization of technical specifications. Within this action program, the Commission took the initiative to establish a set of harmonized technical rules for the structural design of construction works which, in the first stage, would serve as an alternative to the national rules in force in the Member States and, ultimately, would replace them.
mid 80's	The Commission, with the help of a Steering Committee containing Representatives of Member States, conducted the development of the Eurocodes program, which led to the publication of a set of first generation European codes in the 80's.
1989	The Commission and the Member States decided, on the basis of an agreement with CEN (Comité Européen de Normalisation, European Committee for Standardization), endorsed by the SCC, to transfer the preparation and the publication of the Eurocodes to CEN through a Mandate, in order that they would, in the future, have the status of European Standards.
1992-1998	Originally, the Eurocodes were elaborated by CEN as 62 pre-standards (ENVs). Most were published between 1992 and 1998, but, due to difficulties in harmonizing all the aspects of the calculation methods, the ENV Eurocodes included "boxed values" which allowed Members States to

Time	History overview
	choose other values for use on their territory. National Application Documents, which gave the details of how to apply ENV Eurocodes in Member States, were, generally, issued with a country's ENV.
1998	The conversion of ENVs into European standards started in 1998.
2002-2006	Publication of the EN Eurocode Parts is expected between 2002 and 2006.
2010	By 2010 all national rules are to be replaced by the EN Eurocodes. United of Kingdom has approved EN from 2007, the index "BS" has replaced by "BS EN".

Table 1.2-Conversion between peak ground acceleration and earthquake intensity (Appendix K in TCXDVN 375:2006 Part 1[6])

MSK-64 Scale		**MM Scale**	
Intensity	Peak ground acceleration a(g)	Intensity	Peak ground acceleration a(g)
V	0.012-0.03	V	0.03-0.04
VI	>0.03-0.06	VI	0.06-0.07
VII	>0.06-0.12	VII	0.10-0.15
VIII	>0.12-0.24	VIII	0.25-0.30
IX	>0.24-0.48	IX	0.50-0.55
X	>0.48	X	>0.60

Table 1.3-Timeline of the important legal documents in Vietnam

Time	Legal document	Main contents	Promulgator
Before 1996	Construction technical standards	Have standards that do not have laws and regulations. Almost standards are compiled from standards of Former Soviet Union (Russian Federation)	Ministry of Construction
1996	Vietnam Building Code	The regulations shall be required to apply	Ministry of Construction
1999	Circular on guiding the management and application of technical standards in construction works	Allow standards of seven countries and international organizations in Vietnam are applied, such as: England, America, Australia, Japan, ISO,...	Ministry of Construction
2003	Construction Law		National Assembly of Vietnam
2005	Regulations on application for foreign construction standards in construction activities in Vietnam	Allows the application of the standards of other countries and international organizations (non-regulated only 7 countries and international organizations)	Ministry of Construction

Time	Legal document	Main contents	Promulgator
2010	Circular on application for foreign construction standards in construction activities	Abroad construction standards are selected and applied the principle of voluntary and is the construction standard of the countries, international organizations and regional standards organizations.	Ministry of Construction

Table 2.1-Problems in codes of Vietnam, Europe, U.S.

Issue	TCXD 198:1997	TCXDVN 356:2005	TCXDVN 375:2006 (EN 1998-1:2004)	ACI 318-08
Root of code	Former Soviet Union	Russian Fed.	Europe	U.S
Contents for seismic design	Inadequate	No for seismic design	Adequate	Ade.
Design of beam-column joint	No (shear strength)	No (shear strength)	Yes	Yes
Coupling beams	Lintel beam No detailed require.	No	Yes	Yes
Deep beams	No	No	Yes (EN 1992)	Yes (Ch.10)

Table 2.2-Correlation between seismic-related terminology in model codes (Table R1.1.9.1, ACI 318-08 [2])

Code, standard, or resource document and edition	Level of seismic risk or assigned seismic performance or design categories as defined in the Code		
ACI 318-08; IBC 2000, 2003, 2006; NFPA 5000, 2003, 2006; ASCE 7-98, 7-02, 7-05; NEHRP 1997, 2000, 2003	SDC* A, B	SDC C	SDC D, E, F
BOCA National Building Code 1993, 1996, 1999; Standard Building Code 1994, 1997, 1999; ASCE 7-93; 7-95; NEHRP 1991, 1994	SPC† A, B	SPC C	SPC D, E
Uniform Building Code 1991, 1994, 1997	Seismic Zone 0, 1	Seismic Zone 2	Seismic Zone 3, 4

*SDC = Seismic design category as defined in code, standard, or resource document.
†SPC = Seismic performance category as defined in code, standard, or resource document.

Table 2.3-Classification of earthquake level between ACI, EN, TCXDVN

Code	**Seismic Design Category**		
ACI 318-08	SDC: A, B (The lowest seismic cases/hazard)	SDC C (The moderately strong ground shaking)	SDC: D, E, F (The strong ground shaking)
EN 1998-1:2004	DCL $a_g < 0.04g$ (very low seismicity)	DCL $0.04g \le a_g < 0.08g$ (low seismicity)	DCM, DCH $a_g \ge 0.08g$ (strong seismicity)
TCXDVN 375:2006-Part 1	DCL $a_g < 0.04g$ (very low seismicity)	DCL $0.04g \le a_g < 0.08g$ (low seismicity)	DCM, DCH $a_g \ge 0.08g$ (strong seismicity)

where: $a_g = a_{gR}\gamma_I$, a_{gR} and γ_I are given in table in TCXDVN 375:2006-Part 1 [6]

Table 2.4-Table of describing seismic zonation in P.R.China, S.R.Vietnam, U.S.

GB 18306-2001 [20,21] (P.R.China)	**very low**		0.05g **low-to-moderate**						0.25g **strong**		
Intensity Level	-		VI			VII	VII-VIII	VIII		VIII-IX	IX
TCXDVN 375:2006 [6] (EN 1998-1:2004) [4]	**very low**	0.04g **low**			0.08g **strong**						
UBC-1997 [9]			0.05g	0.075g			0.15g	0.2g		0.3g	
Zone	**0**		**1**	**2A**			**2B**	**3**		**4**	
Z=	0		0.075g	0.15g			0.2g	0.3g		0.4g	
	low		**low**	**moderate**				**high**		**very high**	
Peak ground acceleration	0	0.04g	0.05g	0.075g	0.08g	0.1g	0.15g	0.2g	0.25g	0.3g	0.4g

Table 2.5-Proposal for change of TCXDVN 375:2006-Part 1

GB 18306-2001 [20,21] (P.R.China)	**very low**		0.05g **low-to-moderate**						0.25g **strong**		
Intensity Level	-		VI			VII	VII-VIII	VIII		VIII-IX	IX
TCXDVN 375:2006 [6] (EN 1998-1:2004) [4]	**very low**	0.04g **low**			0.08g **strong**						
Proposal for change of TCXDVN 375:2006	**very low**		**0.05g** **low or low-to-moderate**					**0.2g** **strong**			
UBC-1997 [9]			0.05g	0.075g			0.15g	0.2g		0.3g	
Zone	**0**		**1**	**2A**			**2B**	**3**		**4**	
Z=	0		0.075g	0.15g			0.2g	0.3g		0.4g	
	low		**low**	**moderate**				**high**		**very high**	
Peak ground acceleration	0	0.04g	0.05g	0.075g	0.08g	0.1g	0.15g	0.2g	0.25g	0.3g	0.4g

Table 2.6-Proposed solution for Vietnam's seismic design

<table>
<tr><th colspan="3">TCXDVN 375:2006-Part 1</th><th colspan="3">Proposal for TCXDVN 375:2006 Part 1</th></tr>
<tr><td>Ductility class</td><td>Seismic region</td><td>a_g</td><td>Ductility class</td><td>Seismic region</td><td>a_g</td></tr>
<tr><td>DCL</td><td>very low</td><td>$a_g < 0.04g$</td><td>DCL</td><td>very low</td><td>$a_g < 0.05g$</td></tr>
<tr><td rowspan="2">DCL</td><td rowspan="2">low</td><td rowspan="2">$0.04g \leq a_g < 0.08g$</td><td>1st proposal: DCL</td><td rowspan="2">low-to-moderate</td><td rowspan="2">$0.05g \leq a_g < 0.2g$</td></tr>
<tr><td>2nd proposal: DCM</td></tr>
<tr><td rowspan="2">DCM, DCH</td><td rowspan="2">strong</td><td rowspan="2">$a_g \geq 0.08g$</td><td>1st proposal: DCM, DCH</td><td rowspan="2">strong</td><td rowspan="2">$a_g \geq 0.2g$</td></tr>
<tr><td>2nd proposal: DCH</td></tr>
</table>

Table 2.7-Correlation between seismic design categories to seismic zones and recommendation

Code, standard, or resource document and edition	Level of seismic risk or assigned seismic performance or design categories as defined in the Code		
ACI 318-08; IBC 2000, 2003, 2006; NFPA 5000, 2003, 2006; ASCE 7-98, 7-02, 7-05; NEHRP 1997, 2000, 2003	SDC* A, B	SDC C	SDC D, E, F
BOCA National Building Code 1993, 1996, 1999; Standard Building Code 1994, 1997, 1999; ASCE 7-93; 7-95; NEHRP 1991, 1994	SPC† A, B	SPC C	SPC D, E
Uniform Building Code 1991, 1994, 1997	Seismic Zone 0, 1	Seismic Zone 2	Seismic Zone 3, 4
EN 1998-1:2004, TCXDVN 375:2006-Part 1	$a_g < 0.04g$ very low, DCL	$0.04g \le a_g < 0.08g$ low, DCL	$a_g \ge 0.08g$ strong DCM, DCH
1st proposed recommendation by author for TCXDVN 375:2006-Part 1	$a_g < 0.05g$ very low, DCL	$0.05g \le a_g < 0.2g$ low-to-moderate, DCL	$a_g \ge 0.2g$, strong, DCM, DCH
2nd proposed recommendation by author for TCXDVN 375:2006-Part 1	$a_g < 0.05g$ very low, DCL	$0.05g \le a_g < 0.2g$ low-to-moderate, DCM	$a_g \ge 0.2g$, strong, DCH

*SDC = Seismic design category as defined in code, standard, or resource document.

†SPC = Seismic performance category as defined in code, standard, or resource document.

Table 3.1-Major concepts on seismic design and detail in codes

Concept	TCXD 198:1997	TCXDVN 356:2005	TCXDVN 375:2006 (EN 1998-1:2004)	ACI 318-08
Lateral-resisting moment frame	No	No	Yes	Yes
Design of beam-column joint	No (shear strength)	No (shear strength)	Yes	Yes
Ductility level	No	No	Yes	Yes
Strong column weak beam	No	No	Yes	Yes

Table 3.2-Main contents for design and detailing of wide beam-column connections

No.	Content	TCXDVN 356:2005 [8] and TCXD 198:1997 [7] (СНиП 2.03.01-84* and СНиП II-7-81*)	EN 1998-1:2008 [4] (TCXDVN 375:2006 [6])	ACI 318-08 [2] and ACI 352R-02 [29]
1	Wide beam width	TCXD 198:1997 (3.3.2): $b_{w,max} = b_c + 1.5h_w$	$b_w \le \min\{b_c+h_w;\ 2b_c\}$	• ACI 318-08: $b_w = \min\{c_2 + c_2;\ c_2 + 0.75c_1\}$ • ACI 352R-02: $b_b \le \min\{b_c+1.5h_c;\ 3b_c\}$
2	Development length	TCXDVN 356:2005 (8.5.2): $l_{an} = \left(\omega_{an}\dfrac{R_s}{R_b} + \Delta\lambda_{an}\right)d$ but not less than $l_{an}=\lambda_{an}d$ * See Figure 3.8.a	- $l_{bd} = \alpha_1\ \alpha_2\ \alpha_3\ \alpha_4\ \alpha_5\ l_{b,rqd} \ge l_{b,min}$ $l_{b,rqd} = (\phi/4)\ (\sigma_{sd}/f_{bd})$, $f_{bd} = 2.25\eta_1\eta_2 f_{ctd}$ • In tension: $l_{b,min} > \max\{0.3l_{b,rqd};\ 10\phi;100mm\}$ • In compression: $l_{b,min} > \max\{0.6l_{b,rqd};10\phi;100mm\}$ * See Figure 3.8.b	• ACI 318-08: In tension: $l_{dh} = f_y d_b/[65(f'_c)^{1/2}]$ (in.) and not less than of max $\{8d_b$, 6in.$\}$ • ACI 352R-02 (4.5.2.4): $\ell_{dh} = \dfrac{\alpha f_y d_b \text{ (psi)}}{75\sqrt{f_c'}\text{ (psi)}}$ $l^*_{dh} = 0.75l_{dh}$ (headed bar, 4.5.3.3, ACI 352R-02) * See Figure 3.8.c

No.	Content	TCXDVN 356:2005 [8] and TCXD 198:1997 [7] (СНиП 2.03.01-84* and СНиП II-7-81*)	EN 1998-1:2008 [4] (TCXDVN 375:2006 [6])	ACI 318-08 [2] and ACI 352R-02 [29]
				* At exterior connections, beam longitudinal reinforcement that passes outside the column core should be anchored in the core of the transverse beam following the requirements of Section 4.5.2.3 (For Type 1 connections). The critical section for development of such reinforcement should be the outside edge of the beam core.
3	Wide beam's longitudinal reinforcement	Not specific required for the wide beam.	- Not specific required for the wide beam. - Clause 5.6.2.2(1)P: The part of beam	• ACI 318-08: Not specific required for the wide beam. • ACI 352R-02, 3.3.3:

No.	Content	TCXDVN 356:2005 [8] and TCXD 198:1997 [7] (СНиП 2.03.01-84* and СНиП II-7-81*)	EN 1998-1:2008 [4] (TCXDVN 375:2006 [6])	ACI 318-08 [2] and ACI 352R-02 [29]
			longitudinal reinforcement bent in joints for anchorage shall always be placed inside the corresponding column hoops. * These above clauses are not clear because they are needed to point out case of reinforcement passed outside of the column core.	- Type 2 interior wide-beam connections, at least 1/3 of the wide-beam top longitudinal and slab reinforcement that is tributary to the effective width should pass through the confined column core. - Type 2 exterior connections with beams wider than columns, at least 1/3 of the wide-beam top longitudinal and slab reinforcement that is tributary to the effective width should be anchored in the column core.

No.	Content	TCXDVN 356:2005 [8] and TCXD 198:1997 [7] (СНиП 2.03.01-84* and СНиП II-7-81*)	EN 1998-1:2008 [4] (TCXDVN 375:2006 [6])	ACI 318-08 [2] and ACI 352R-02 [29]
4	Transverse reinforcement through column core to confine beam longitudinal bars passing outside column core	No	No	• ACI 318-08: Have requirement, but not clear about design and detail of this bar. • ACI 352R-02: No

No.	Content	TCXDVN 356:2005 [8] and TCXD 198:1997 [7] (СНиП 2.03.01-84* and СНиП II-7-81*)	EN 1998-1:2008 [4] (TCXDVN 375:2006 [6])	ACI 318-08 [2] and ACI 352R-02 [29]
5	Diameter of beam reinforcement passed outside colomn core	Not required	Not required. There only for reinforced bars passing through beam-column joints: - Clause 5.6.2.2(2)P: To prevent bond failure, the diameter of beam longitudinal bars passing through beam-column joints, d_{bL}, shall be limited in accordance with the following expressions: For interior joints: $\frac{d_{bL}}{h_c} \le \frac{7,5 f_{ctm}}{\gamma_{Rd} f_{yd}} \frac{1+0,8\nu_d}{1+0,75 k_D \rho'/\rho_{max}}$	• ACI 318-08: Not required. • ACI 352R-02, 4.5.5 Beam longitudinal reinforcement passing outside the joint core should be selected: $\frac{h_{(\text{column})}}{d_{b\ (\text{beam bars})}} \ge 24 \frac{f_y}{60{,}000} \ge 24\ (\text{psi})$

No.	Content	TCXDVN 356:2005 [8] and TCXD 198:1997 [7] (СНиП 2.03.01-84* and СНиП II-7-81*)	EN 1998-1:2008 [4] (TCXDVN 375:2006 [6])	ACI 318-08 [2] and ACI 352R-02 [29]
			For exterior joints: $$\frac{d_{bL}}{h_c} \le \frac{7{,}5 f_{ctm}}{\gamma_{Rd} f_{yd}} \left(1+0{,}8 \nu_d\right)$$	
6	Transverse reinforcement of wide beam	Not specific required for the wide beam.	Not specific required for the wide beam.	• ACI318-08: Not required • ACI 352R-02: 4.6.2-Beam transverse reinforcement: Computed beam shear stresses < $2(f'_c)^{1/2}$ (psi), the maximum spacing of transverse reinforcement should be the least of: - 1/2 the effective wide beam depth, - eight times the longitudinal bar diameter,

No.	Content	TCXDVN 356:2005 [8] and TCXD 198:1997 [7] (СНиП 2.03.01-84* and СНиП II-7-81*)	EN 1998-1:2008 [4] (TCXDVN 375:2006 [6])	ACI 318-08 [2] and ACI 352R-02 [29]
				- or 24 times the stirrup bar diameter. A minimum of four stirrup legs should be provided.
7	Transverse beam	Not required	Not required	• ACI318-08: Not required • ACI 352R-02: 3.3.3-For Type 2 exterior wide-beam connections, the transverse beam should be designed to resist the full equilibrium torsion from the beam and slab bars anchored in the spandrel beam within the slab effective width, b_e, following the requirements of Section 11.6

No.	Content	TCXDVN 356:2005 [8] and TCXD 198:1997 [7] (СНиП 2.03.01-84* and СНиП II-7-81*)	EN 1998-1:2008 [4] (TCXDVN 375:2006 [6])	ACI 318-08 [2] and ACI 352R-02 [29]
				of ACI 318-02. The spacing of torsion reinforcement in the transverse beam should not exceed the smaller of p_h/16 and 6 in. (150 mm), where p_h is the perimeter of centerline of the beam outermost closed transverse torsion reinforcement.
8	Shear strength	Not required (for both conventional beam-column joints and wide beam-column joints)	- Design for DCM (5.4, 5.4.3.3): Only requirements about details of the horizontal confinement reinforcement in joints and vertical bar in column. - Design for DCH (Ductility class	• ACI318-08: Not specific required for the wide beam. • ACI 352R-02: - Joint shear for Type 1 and Type 2 connections:

No.	Content	TCXDVN 356:2005 [8] and TCXD 198:1997 [7] (СНиП 2.03.01-84* and СНиП II-7-81*)	EN 1998-1:2008 [4] (TCXDVN 375:2006 [6])	ACI 318-08 [2] and ACI 352R-02 [29]
			high) (5.5.2.3, 5.5.3.3): • In interior joints: $V_{jhd} = \gamma_{Rd} (A_{s1} + A_{s2}) f_{yd} - V_C$ $V_{jhd} \le V_n = \eta f_{cd} b_j h_{jc} (1-\nu_d/\eta)^{1/2}$ • In exterior joints: $V_{jhd} = \gamma_{Rd} A_{s1} f_{yd} - V_C$ $V_{jhd} \le V_n = 0.8\eta f_{cd} b_j h_{jc} (1-\nu_d/\eta)^{1/2}$ * $b_c<b_w$: $b_j = \min\{b_w;(b_c+0.5h_c)\}$	$\phi V_n \ge V_u$ where $\phi=0.85$ The nominal shear strength of joint, V_n, is $V_n = \gamma \sqrt{f_c'} b_j h_c$ (psi) b_j should not exceed the smallest of $(b_b+b_c)/2$ and $b_b + \sum \frac{mh_c}{2}$ and b_c (m depends on eccentricity between the beam centreline and the column centroid, m=0.3 and 0.5)

* Definitions in ACI 352R-02 [29]:

- A beam-column joint is defined as that portion of the column within the depth of the deepest beam that frames into the column. Throughout this document, the term joint is used to refer to a beam-column joint.

- A connection is the joint plus the columns, beams, and slab adjacent to the joint.

- A transverse beam is one that frames into the joint in a direction perpendicular to that for which the joint shear is being considered.

- Type 1-A Type 1 connection is composed of members designed to satisfy ACI 318-02 strength requirements, excluding Chapter 21, for members without significant inelastic deformation.

- Type 2-In a Type 2 connection, frame members are designed to have sustained strength under deformation reversals into the inelastic range.

Table 3.3-Existing wide beam-column connections in some regions in Vietnam

Peroid	Intensity	Types of Building and structure
Before 2006	Low seismic regions (Scale VI-MSK 64): a_g = 0.03-0.06(g): No seismic design, only mitigated detail measures or no seismic detail	- High rise buildings - Wide beam
2006 up to now (TCXDVN 375:2006-Part 1)	Strong seismic regions $a_g \geq 0.08$(g): Seismic design and detail (Some zones in Low seismic regions)	- High rise buildings - Wide beam

Table 3.4-Beam-core wall connections

Considerations	Hinge joint
EN 1998-1:2004, ACI 318-08	Violated provisions for special moment frame, design of DCM, DCH
Primary seismic members	Not allowed
Secondary seismic members	Can be used
Lateral force resisting frame	Low capacity (lateral force transfer mechanism is restrained by hinge connections)
Core wall	- Performance of core wall is less, leads to waste. - Control to provide enough strength and detailing requirement
Others	- Roughness of the support interface - Force transfer mechanism - Flexibility of structure - Serviceability of building

Table 3.5-Drift and displacement ductility ratios

Specimen	Shape ratios				Drift and displacement ratios			Source
	b_b/h_b	b_c/h_c	b_b/b_c	h_s/h_b	δ_{yo} (%)	δ_u (%)	μ	
EL	2.7	1	2.0	1.0	2.4	4.9	2.0	[39]
EU	2.0	1	2.0	1.0	3.1	4.6	1.5	[39]
EWB	4.2	1	3.0	1.0	1.5	5.4	3.6	[26,39]
EWB-1	2.8	1	2.4	1.3	1.3	>5.0	>3.8	[39,42]
EWB-2	2.8	1	2.4	2.0	1.3	>4.7	>3.6	[39,42]
EWB-3	3.1	0.6	3.1	1.5	1.2	>2.9	>2.4	[39,42]
1	2.8	1	2.43	1.3	2.5	>5.0	>2.0	[16,39]
2	2.5	1	2.14	1.3	1.6	>5.0	>3.1	[16,39]
3	2.8	1	2.43	1.3	1.8	>5.0	>2.8	[16,39]
4	2.8	1	2.43	1.3	1.9	>5.0	>2.6	[16,39]
WB-1	1.5	1	0.9		1.0	>2.0	>2.0	[34]
WB-2	3.0	1	1.8		1.4	>4.5	>3.2	[34]
WB-3	4.5	1	2.7		1.4	>3.9	>2.8	[34]
WB-4	6.0	1	3.6		1.4	>4.0	>2.9	[34]
WF-1	3.0	1	1.8		1.2	>5.0	>4.2	[34]

Specimen	Shape ratios				Drift and displacement ratios			Source
	b_b/h_b	b_c/h_c	b_b/b_c	h_s/h_b	δ_{yo} (%)	δ_u (%)	μ	
WF-2	3.0	1	1.8		1.0	>4.9	>4.9	[34]
WF-3	3.0	1	1.8		0.9	>2.0	>2.2	[34]
WF-4	3.0	0.25	3.6		0.8	>2.0	>2.5	[34]
IWB1	2.7	0.3	2.7		0.5	>4.0	>8	[36]
IWB2	2.7	3.0	0.9		0.5	>3.0	>6	[36]
IWB3	2.7	0.3	2.7		0.5	>5.0	>10	[36]
EWB1	2.7	0.3	2.7		0.5	2.5	5.0	[37]
EWB2	2.7	3.0	0.9		0.5	>3.0	>6.0	[37]
EWB3	2.7	0.3	2.7		0.5	2.5	5.0	[37]
IL	2.7	1	1.8	1	1.2	6.5	5.4	[38]
IU	2.0	1	1.7	1	1.3	6.0	4.6	[38]
1st Spec.	6	1	4.8		0.8	>4.0	>5.0	[40]
2st Spec.	6	1	4.8		1.0	>4.0	>4.0	[40]
WBB-I1	6	1	4.8		0.5	>3.5	>7.0	[41]
WBB-I2	6	1	4.8		1.0	>3.5	>3.5	[41]
WBB-I3	6	1	4.8		1.0	>3.5	>3.5	[41]

Table 3.6-Calculation for joint shear strength

Specimen	Interior/ Exterior	Seismic region	V_{jhd} (tons) (EN, See Table 3.2)	V_n (tons) (EN, See Table 3.2)	V_{jhd}/V_n	Source
1	Exterior	High	82	117	0.70	[16]
2	Exterior	High	72	117	0.61	[16]
3	Exterior	High	79	117	0.67	[16]
4	Exterior	High	103	117	0.88	[16]
EWB-1	Exterior	High	81	126	0.64	[42]
EWB-2	Exterior	High	88	127	0.69	[42]
EWB-3	Exterior	High	114	230	0.50	[42]
IWB1	Interior	Moderate	210	1,175	0.18	[36]
IWB2	Interior	Moderate	357	343	1.04	[36]
IWB3	Interior	Moderate	212	952	0.22	[36]
EWB1	Exterior	Moderate	270	938	0.28	[37]
EWB2	Exterior	Moderate	267	273	0.98	[37]
EWB3	Exterior	Moderate	269	761	0.35	[37]

Table 3.7-Brief summary on experimental tests for wide beam-column joints

Source	Seismic zone	Interior specimen	Exterior specimen	Conclusion
Gentry and Wight [16]	High	- Analysis 9 test of Kajima Inst. - Analysis of 1 test of U.C at Berkeley	4	- Story drift ratio reached 1.5%-2.5% at first yield. - Wide beam-column exterior and interior connections can be used in strong seismic zones if they are detailed correctly (wide beam width is not excessive by requirement). - Torsion demand on the transverse beam must be controlled
Benavent-Climent [26]	Moderate	1 (IWB)	1 (EWB)	- Drift ratio: 1.5%-EWB, 3%-IWB. - The beam-column joints do not fail.
Hatamoto et al. [34]	High	8		- Story drift ratio reached average 1.1% at first yield. - Experimental test results showed that the energy dissipating capacity of wide beam-column joints is almost equal to conventional beam-column connection (LaFave and Wight, 1999 [41], 2001). - The amount of beam longitudinal bars outside joint core should be

Source	Seismic zone	Interior specimen	Exterior specimen	Conclusion
				limited in terms of torsional stress in outside beam portion. - Sufficient transverse reinforcement should be provided in the outside beam part not only to improve torsional rigidity after cracking but also to provide adequate anchorage for the beam rebars placed outside column core.
Popov et al. [34]	Moderate	0	1	- Story drift ratio reached 2% at first yield strain.
Kulkarni and Li [36]	Moderate	3	0	- Drift ratio reached 1% (IWB1), 1.5% (IWB3), 2% (IWB3) at first yield. - Transverse beam is a critical issue in the design of wide-beam systems. - Joint shear requirement could be relaxed for wide beam column frames.
Li and Kulkarni [37]	Moderate	0	3	- Beam bars yielded corresponding to a drift ratio of 1.5% (EWB1), 2% (EWB2), 1.5% (EWB3). - It can generally attain their strength and deformation capacity. - Design and detailing of the transverse beam is a critical issue, which

Source	Seismic zone	Interior specimen	Exterior specimen	Conclusion
				needs to be carefully addressed. - Requirement of beam shear reinforcement can be relaxed (due to larger section of the wide beam, shear stresses in the beam transverse reinforcement were very low) - Paulay et al. (1978) also recommended that at least three-fourths of the beam longitudinal bars should pass through the column core.
Benavent-Climent et al. [38]	Moderate	2	0	- Average drift ratios at first yielding: 2.1% and 3.4% (Design of the specimens did not meet the requirements of ACI-ASCE Committee 352) - The ultimate energy dissipation capacity is about 9, which is about one fourth of the value recommended for providing adequate seismic performance.
Benavent-Climent et al. [39]	Moderate	0	2	- Average drift ratios at first yielding: 2.2% - Both specimens behaved as a strong column-weak beam mechanism. - The ultimate energy dissipation capacity is about 4, which is about one

Source	Seismic zone	Interior specimen	Exterior specimen	Conclusion
				eighth of the value recommended for providing adequate seismic performance. This type of structure located in earthquake prone areas should be seismic retrofitted.
Stehle et al. [40]	High	2	0	- Maximum drift ratio at first yield strain: 2% and 4% - Connection could be suitable for use in regions of high seismicity as part of a moment-resisting frame. Limits on beam width in regions of high seismicity could be removed from current design codes if the unique detailing strategy is used. - It would allow the use of the more economically efficient band beam flooring system in regions of high seismicity as part of the primary lateral-load-resisting system.
Siah et al. [41]	Moderate	3	0	- First yielding of reinforcement was observed at a drift ratio of 1.6% and 0.8% for specimens (one specimen, WBB-I1, was not observed because of the severe torsion cracks that led to premature failure of the

Source	Seismic zone	Interior specimen	Exterior specimen	Conclusion
				connection) - Practicing engineer has more options regarding the following two issues: (i) Debonding outside beam reinforcement if necessary, (ii) Using the wide beam system as a gravity load resisting system in a seismic region, as a secondary lateral load resisting system, or even a primary lateral load resisting system, depending on the expected level of seismicity.
LaFave and Wight [42]	High	0	3	- The wide beam connections started to exhibit flexural beam yielding at drifts of less than 1-0.5%, and they reached their design strengths by 2 percent drift. - The wide beam connections performed well, even when b_w/b_c was greater than three and when more than two-thirds of the wide beam flexural reinforcement was anchored outside the column. core (in the spandrel beam core).

Table 4.1-Coupling beams in codes of Vietnam, Europe and U.S.

Content	TCXD 198:1997	TCXDVN 356:2005	TCXDVN 375:2006 (EN 1998-1:2004)	ACI 318-08
Coupling beams	Use term of "Lintel beam" No coupling beam	No	1 option	2 options
Provisions for design	No	No	Yes	Yes
Provisions for detail	No	No	Yes	Yes

Table 4.2-Brief summary on coupling beams

in codes of Vietnam, Europe and U.S.

Content	**TCXD 198:1997**	**TCXDVN 356:2005**	**TCXDVN 375:2006 (EN 1998-1:2004)**	**ACI 318-08**
Design for coupling beams	Not guide	No	$l/h < 3$ $V_{Ed} \le 2A_{si}f_{yd}\sin\alpha$	$l/h < 4$ $V_n = 2A_{vd}\, f_y\sin\alpha$ $\le 10A_{cw}(f'_c)^{1/2}$ $V_u \le \phi V_n$
Dimension of cage	No	No	b and h $\ge b_w/2$	$b \ge b_w/2$; $h \ge b_w/5$
Diagonal bars	No	No	No.bars=calculated	Min=4 bars
Angle bend of diag. bars	No	No	No	No
Anchorage of horiz. bars	No	No	Not anchored or only 150mm	Not develop
Transverse reinf. for diag.bars	Spiral stirrup	No	Tie stirrup	Tie stirrup
Options for detail	1	No	1	2

Table 5.1-Deep beams in codes of Vietnam, Europe, U.S.

Concept	TCXD 198:1997	TCXDVN 356:2005	TCXDVN 375:2006 (EN 1998-1:2004)	ACI 318-08
Strong column weak beam	No	No	Yes	Yes
Deep beams	No	No	Yes*	Yes**
STM	No	No	Yes*	Yes**

Notes:

*: EN 1992-Eurocode 2-Design of concrete structures

EN 1992 still not adopted in Vietnam

**: Chapter 10, not in Chapter 21-Earthquake-resistant structures.

Table 5.2-Brief summary of design on deep beams

in codes of Vietnam, Europe, U.S.

Content & Criteria	**TCXD 198:1997**	**TCXDVN 356:2005**	**TCXDVN 375:2006 (EN 1998-1:2004)**	**ACI 318-08**
Design of deep beams	No	No	No Only in EN 1992* Non seismic zones	No Only in Ch.10** Non seismic zones
Strong column and weak beam	No	No	$\sum M_{Rc} \geq 1.3 \sum M_{Rb}$ (EN 1998-1)	$\sum M_{nc} \geq (6/5) \sum M_{nb}$ If not: Another lateral load resisting elements should be provided; and column is not designed as part of seismic-force-resisting system

Notes:

*: EN 1992-Eurocode 2-Design of concrete structures

EN 1992 still not adopted in Vietnam

**: Chapter 10, not in Chapter 21-Earthquake-resistant structures.

Table 5.3-Deep beams and its problems

Content	Existed issues	Future
Model for experimental test	- For single and two span deep beams only. Bearing plate is used more than column stub. Width of DB ≅ and < 500mm. - A little tests subjected to reserved cyclic loads	- Test model come to practice - Use deep beams in seismic design
Continuous deep beams	- Treated as simply supported deep beams, except some additional requirements - No definition in EN 1998-1:2004, ACI 318-08	- Conservative or not?
Analysis model	- Quite many models and quite difficulties - Only axial force, not mentioned on moment…	- Easier model - Accuracy
Deep beams in seismic regions	- Vietnam: Design of the deep beams as transfer beams shall be prohibited in seismic regions with $a_g \geq 0.08g$ (strong seismic regions). - In very low to low seismic regions: Recommendation: Only for low and medium rise buildings	- Safety. - Conservative or not?

Table 5.4-Mode of failure with difference span-depth ratios [80]

Span-Depth Ratio	Cracking Load	Failure Load	Mode of Failure
12	340 (kN)	490 (kN)	Shear failure
10	350 (kN)	516 (kN)	Shear failure
8	362 (kN)	556 (kN)	Flexural-Shear failure
6	370 (kN)	600 (kN)	Flexural-Shear failure
4	382 (kN)	640 (kN)	Flexural failure
3	386 (kN)	660 (kN)	Flexural failure
2	390 (kN)	660 (kN)	Flexural failure

Table 5.5-Mode of failure with difference width ratios [80]

Width Ratio	Cracking Load	Failure Load	Mode of Failure
0.5	292 (kN)	464 (kN)	Flexural-Shear failure
1	330 (kN)	540 (kN)	Flexural-Shear failure
2	370 (kN)	600 (kN)	Flexural-Shear failure
3	378 (kN)	630 (kN)	Flexural-Shear failure
4	380 (kN)	636 (kN)	Flexural failure
5	380 (kN)	640 (kN)	Flexural failure

Table 5.6-Comparison of results between proposed formulas based on box foundation analogy and FEM [81]

	M_b ($\times 10^3$ kN m)		T ($\times 10^3$ kN)	
h_b (m)	Design formulas	FEM analysis	Design formulas	FEM analysis
1.6	1.19	1.17	4.41	4.42
2.4	3.24	3.13	4.43	4.45
3.2	6.36	6.10	3.95	3.98

Note: FEM=finite element method.

Table 5.7-Earthquake records [84]

Earthquake	Peak acceleration (g)	Direction of excitation
Minor	0.02–0.06	Unidirectional and bidirectional
Moderate	0.08–0.14	Bidirectional
Major	0.15–0.20	Bidirectional
Supermajor	0.25–0.34	Bidirectional

Table 5.8-Observed damage [84]

Earthquake	Observations and types of damage	Condition
Minor	No noticeable shaking, and some small cracks barely noticeable	Serviceable condition
Moderate	Observable vibrations and 8 new cracks at stories above the transfer	Serviceable condition
Major	Significant vibrations and 56 new cracks at story above the transfer and at middle and upper stories	Moderately damaged, no collapse and requiring repai
Supermajor	Structural integrity was destroyed at level right above the transfer	Collapse

Table 5.9-Relationship between structural damage and story drift [84]

Description of structural damage	Story drift
Small cracks on columns in frames	1/1,000–1/1,300
A few number of small cracks on shear walls	1/1,100–1/1,200
Many through-cracks on shear walls	1/300–1/700
Shear walls damaged with concrete crushed and reinforcement exposed	1/80–1/200

Table 5.10-Observed damage [85]

Maximum acceleration of El-Centro Earthquake (% *g*)	Observations and types of damage	Transfer plate condition
2–4	New cracks were not observed	Serviceable condition
8–16	New cracks were observed at the bottom of the transfer plate	Serviceable condition
32	New cracks cutting through the transfer plate	Moderately damaged, no collapse, and requiring repair
64	Structure was severely damaged at the bottom of the transfer plate	Severely damaged and no collapse

Table 5.11-Relationship between structural damage and story drift [85]

Description of transfer plate damage on the transfer plate	Interstory drift at 2/F
None	1/1,500
Slight	1/750
Moderately	1/360
Severely	≤1/180

Table 5.12- Summary of maximum inter-story drift [85]

Maximum acceleration of El-Centro Earthquake record (%)	Maximum interstory drift (mm)		Maximum interstory drift ratio	
	$1/F$	$2/F$	$1/F$	$2/F$
2	0.24	0.18	1/5,729	1/3,972
4	0.62	0.47	1/2,218	1/1,521
8	1.15	0.95	1/1,196	1/753
16	2.49	1.98	1/552	1/361
32	5.69	3.92	1/242	1/182
64	12.15	7.88	1/113	1/91

Table 5.13-Existing transfer structures in some regions in Vietnam

Peroid	**Intensity**	**Types of Building and structure**
Before 2006	Low seismic regions (Scale V, VI-MSK 64): No seismic design, only mitigated detail measures or no seismic detail	- High rise buildings - Transfer structures
2006 up to now (TCXDVN 375:2006-Part 1)	Strong seismic regions $a_g \geq 0.08$(g): Seismic design and detail (Some zones in Low seismic regions)	- High rise buildings - Transfer structures

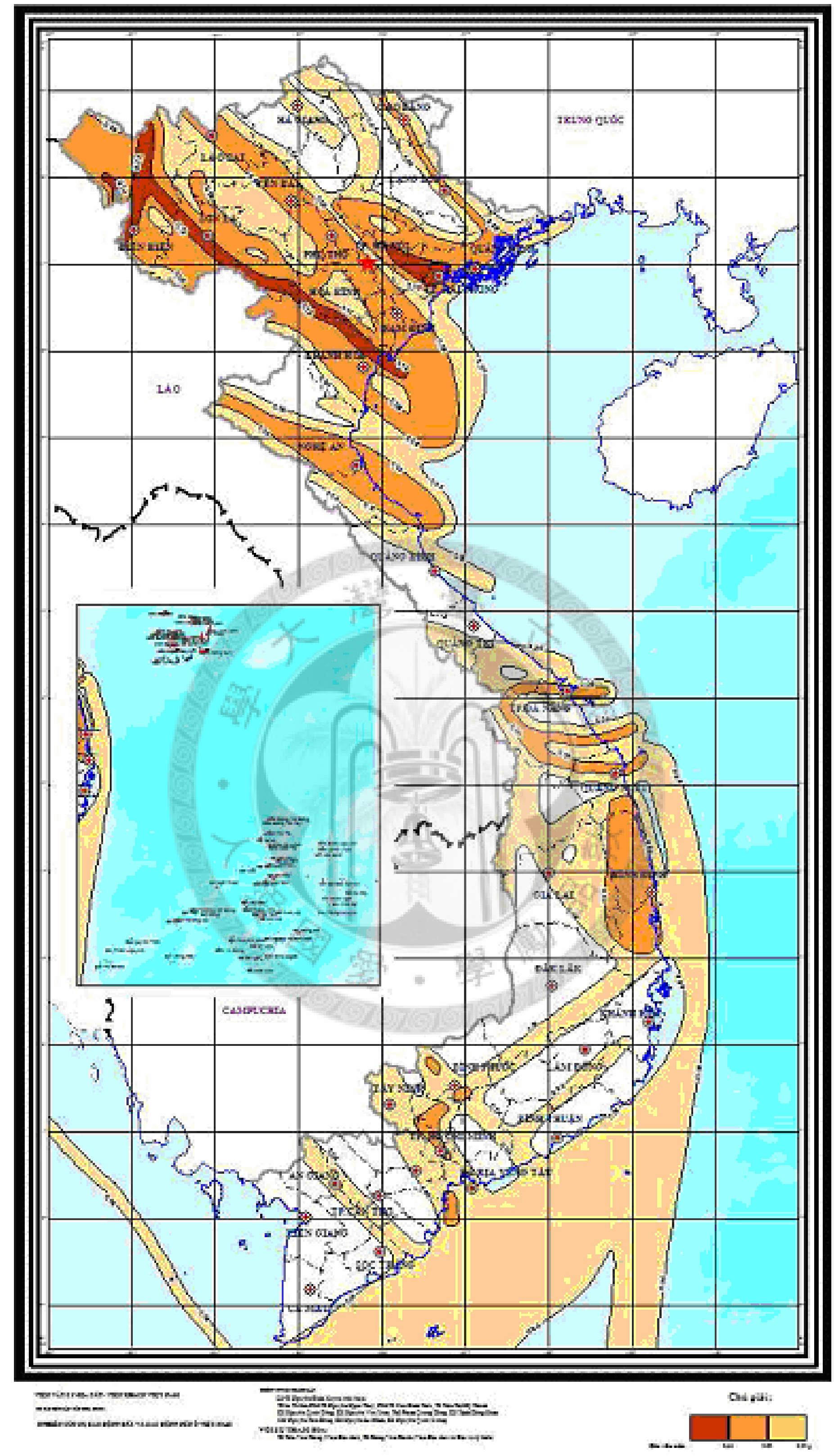


Figure 1.1-Seismic zonation map of Vietnam
(TCXDVN 375:2006-Part 1 [6])

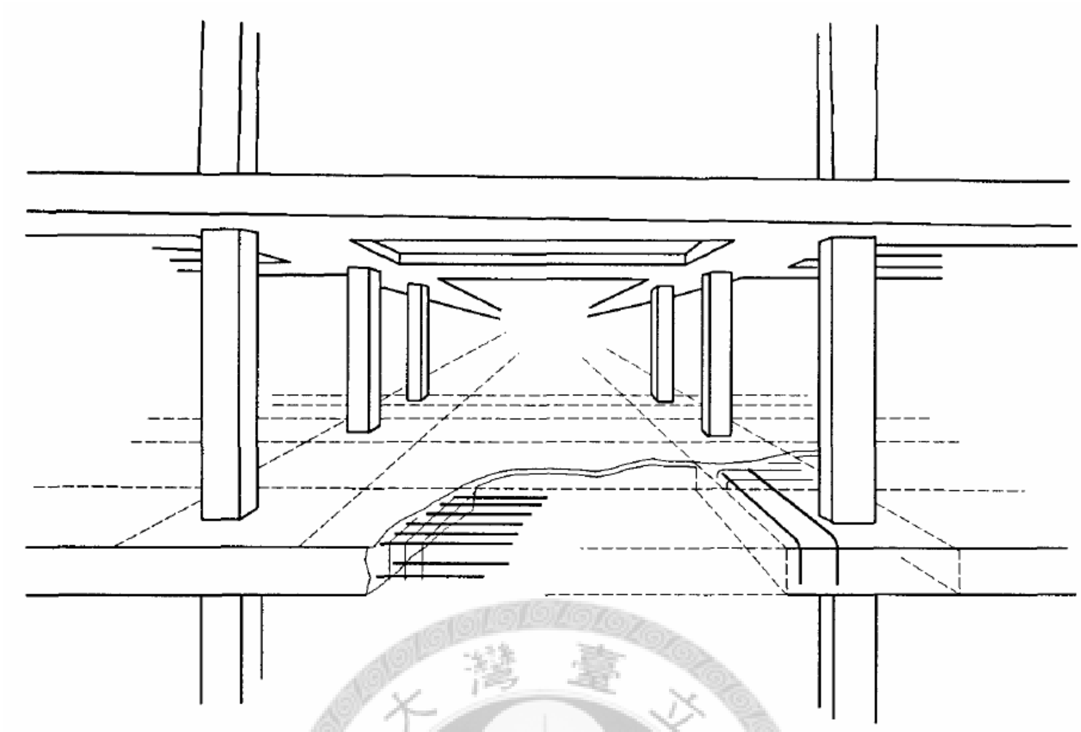


Figure 2.1-An example for reinforced concrete frame using wide beam-column connections (slab not shown for clarity)

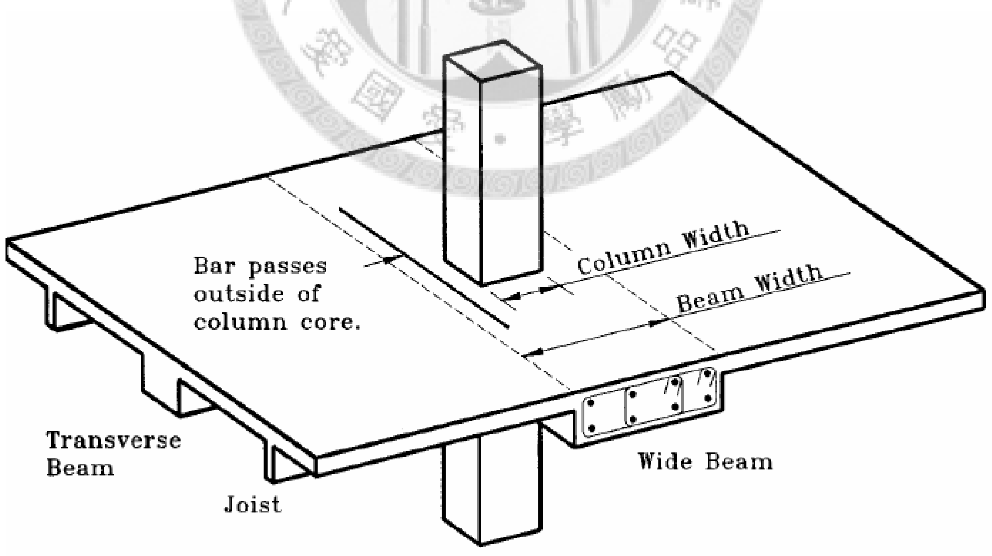


Figure 2.2-Wide beam-column connection, part of the beam longitudinal bars passed outside of the column core (transverse beam is not wide beam in this case)

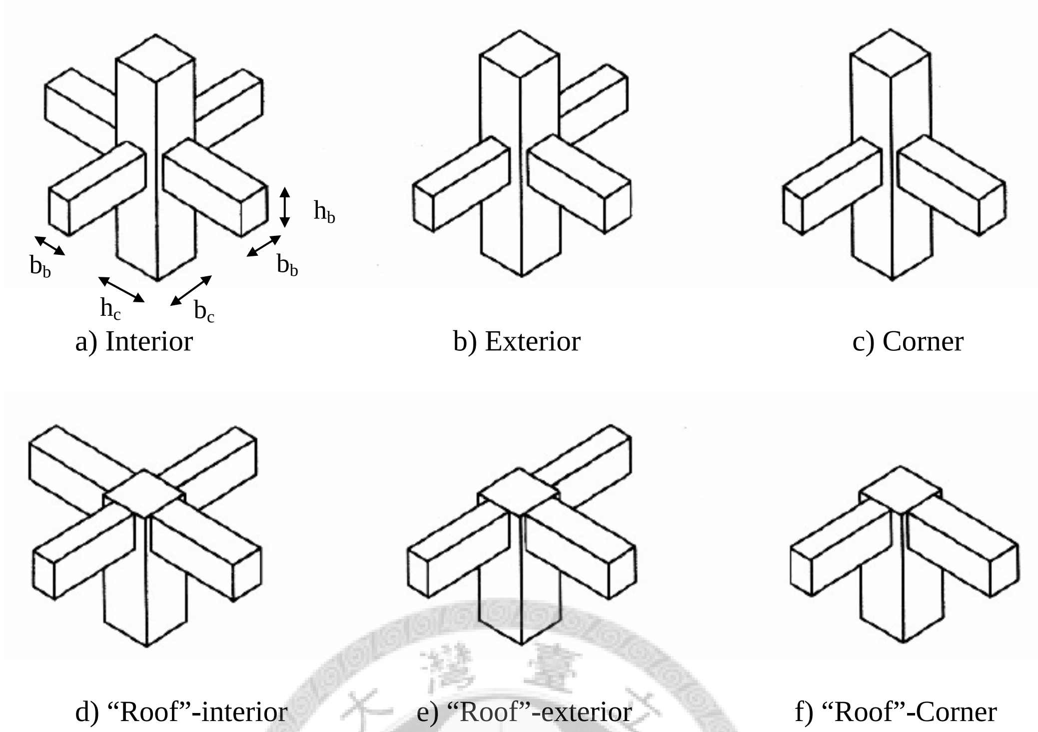


Figure 2.3-Typical beam-column connections as beam width less than and equal column section width/depth (slabs not shown for clarity): $b_b \leq b_c$ and $b_b \leq h_c$

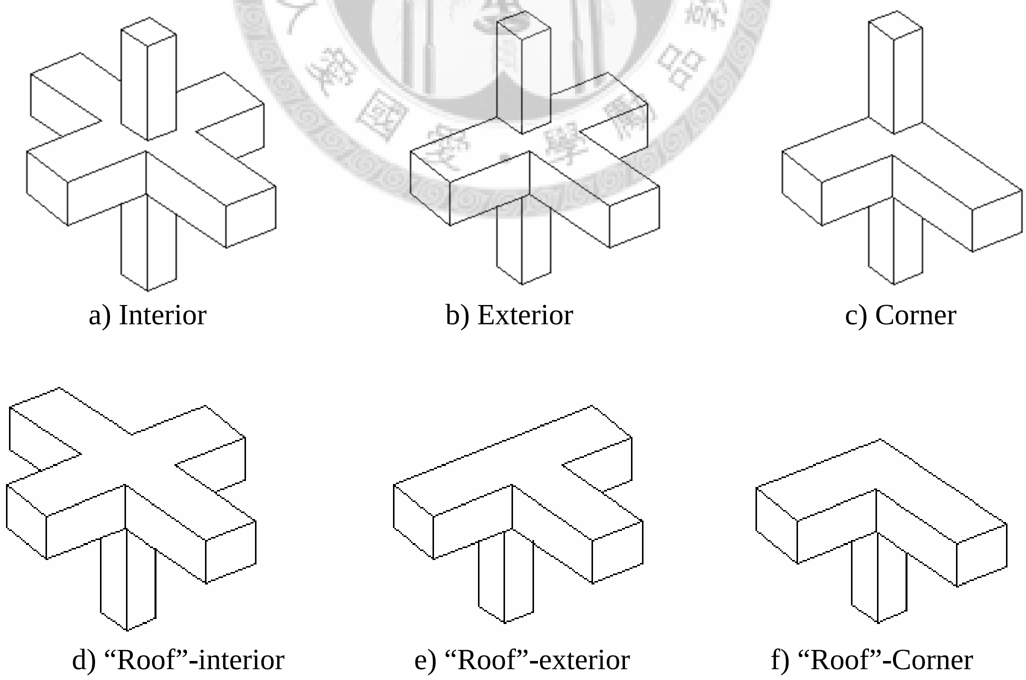


Figure 2.4-Typical wide beam-column connections (slabs not shown for clarity)

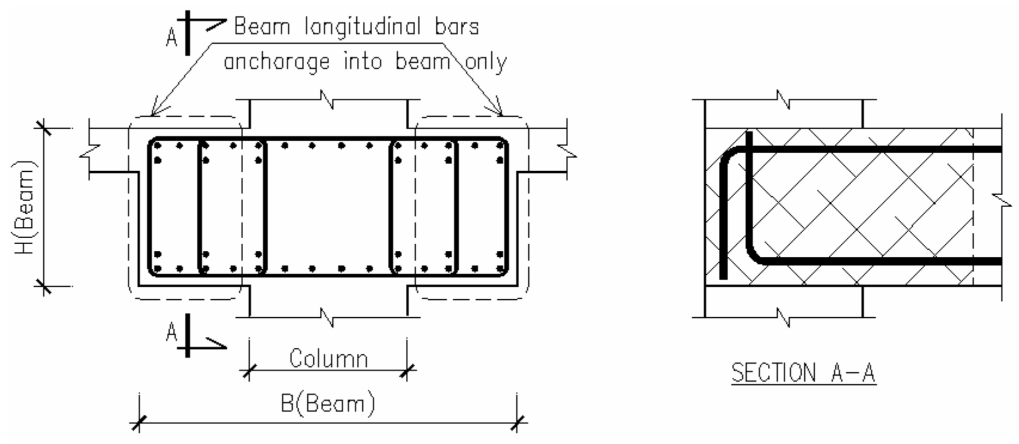


Figure 2.5-Exterior joint: Anchoring of reinforcement in wide beam (reinforcement for column and others not shown for clarity)

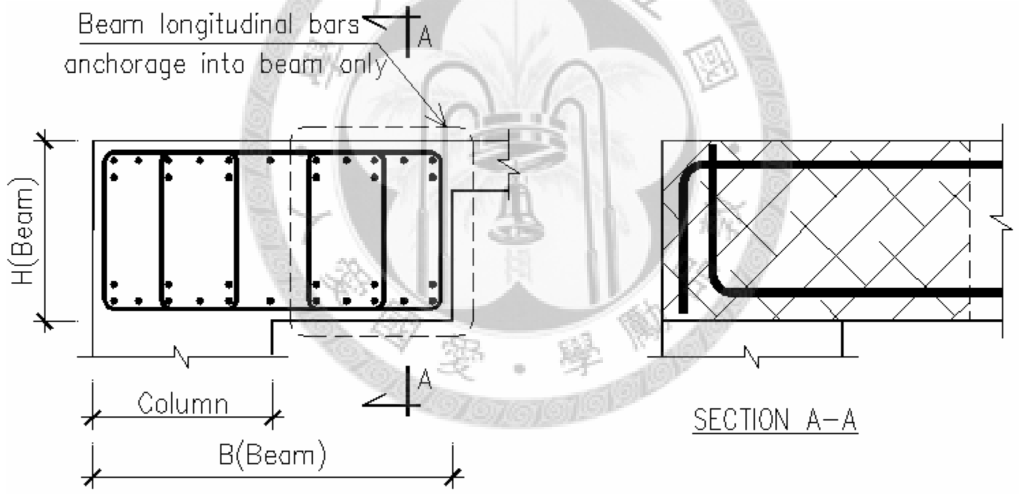
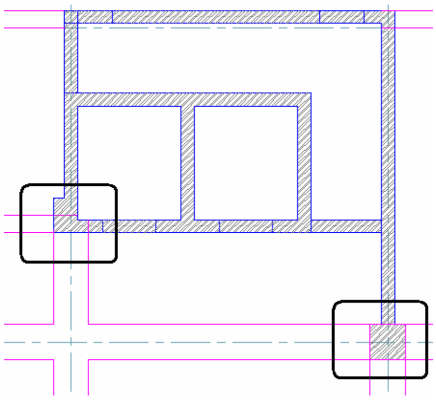
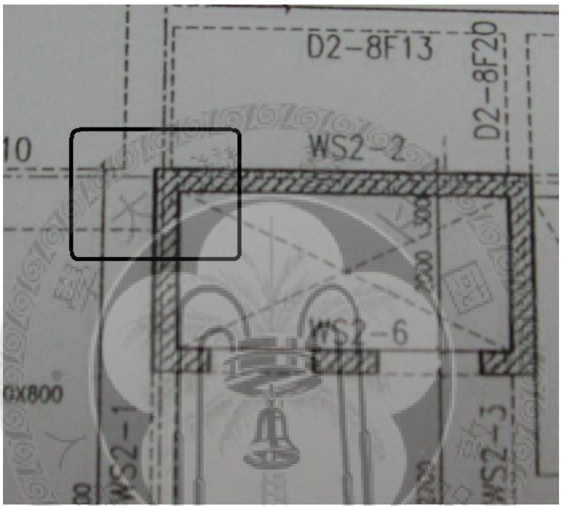


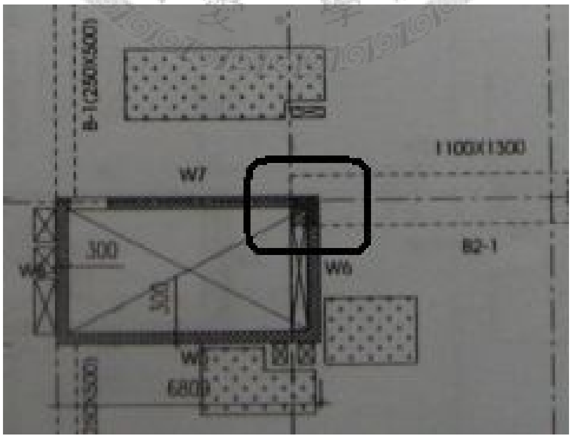
Figure 2.6-Roof-corner joint: Anchoring of reinforcement in wide beam (reinforcement for column and others not shown for clarity)



a)

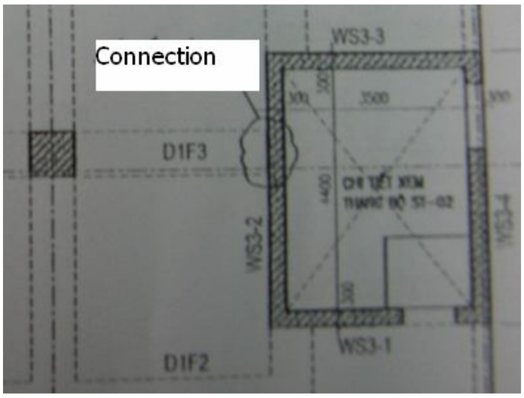


b)

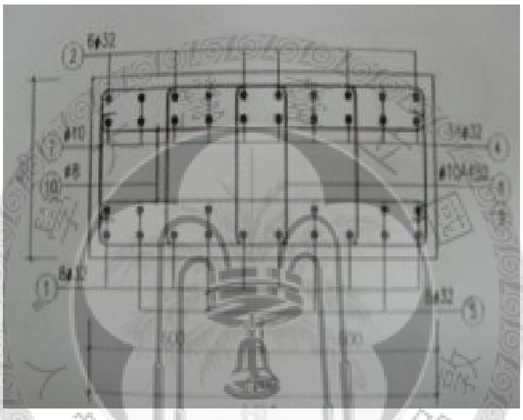


c)

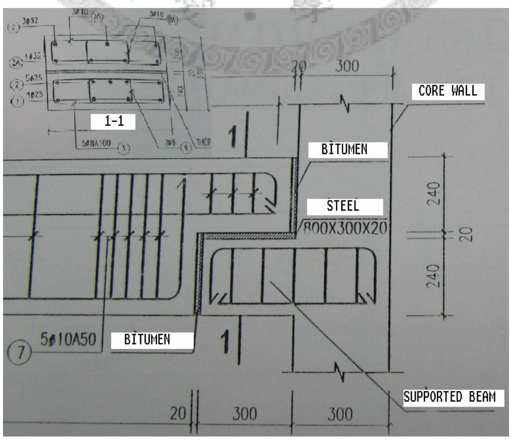
Figure 2.7-Plan view of connection of beam with core wall at expanded wall corner (a) and no fully expanded wall corner (b, c)



a) Plan view

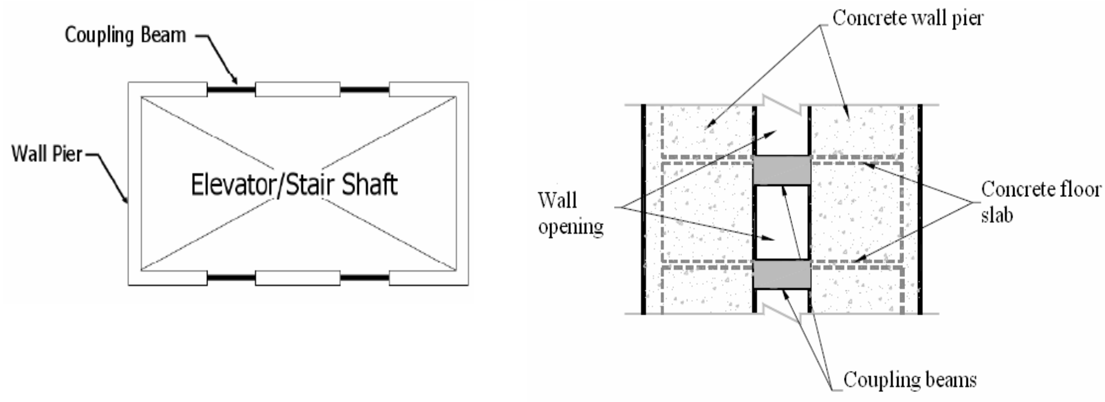


b) "Rigid connection"



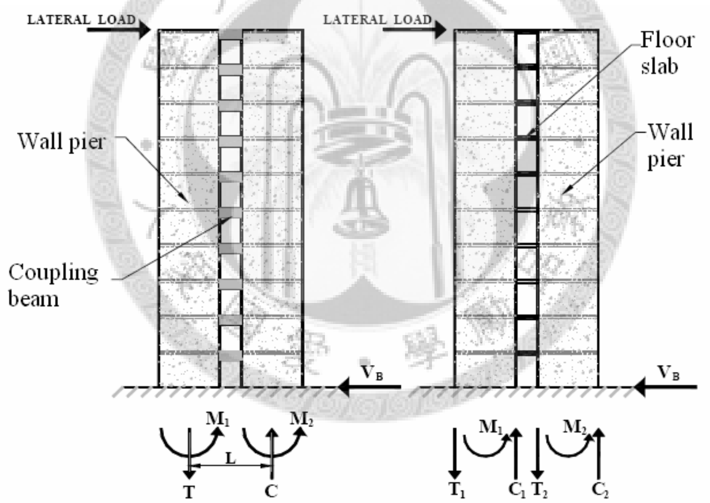
c) "Hinge connection"

Figure 2.8-Beam-core wall joint at no expanded wall and layout of reinforcement in beam D1F3



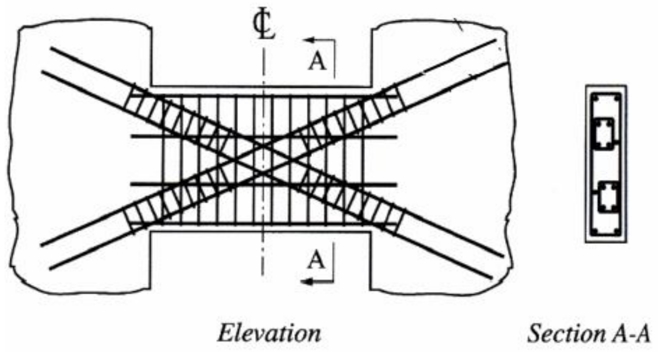
a) **b)**

Figure 2.9-Core wall and coupling beams: a) Plan view; b) Elevation for typical members

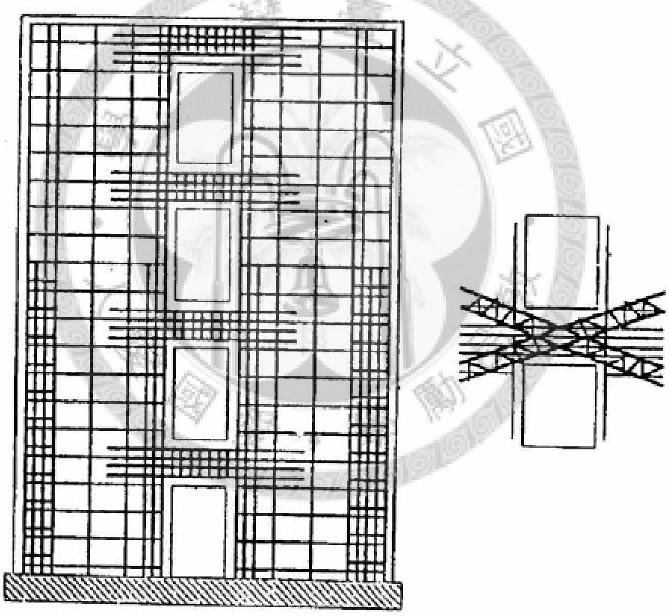


a) **b)**

Figure 2.10-Reaction mechanisms to lateral loads of (a) a coupled wall pier system and (b) an uncoupled wall pier system

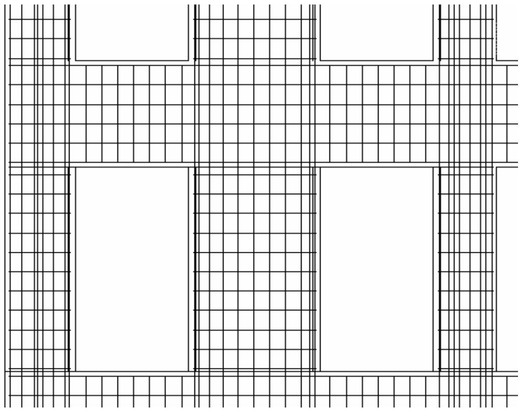


a) Detail in EN, ACI codes

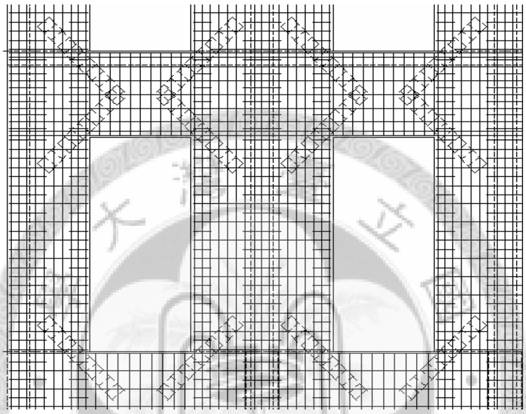


b) Detail in TCXD 198:1997 [7]

Figure 2.11-Schematic of diagonal bars in coupling beams

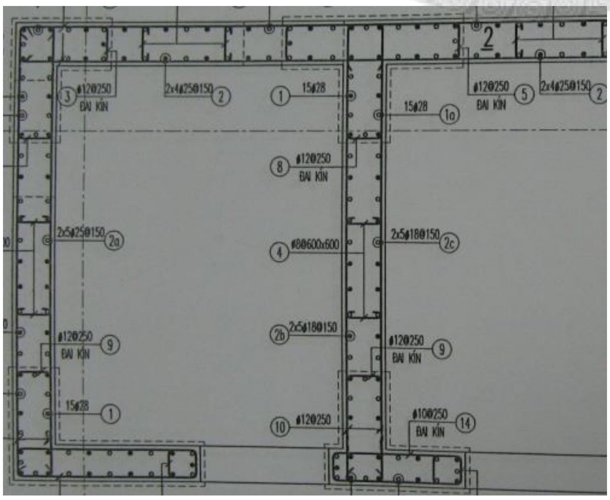


a) Without diagonal bars

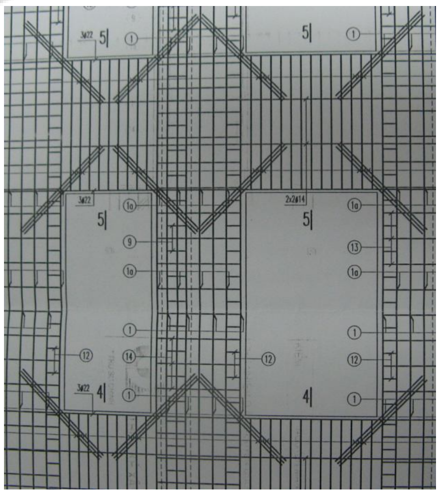


b) With diagonal bars

Figure 2.12-Arrangement of diagonal bars in coupling beams

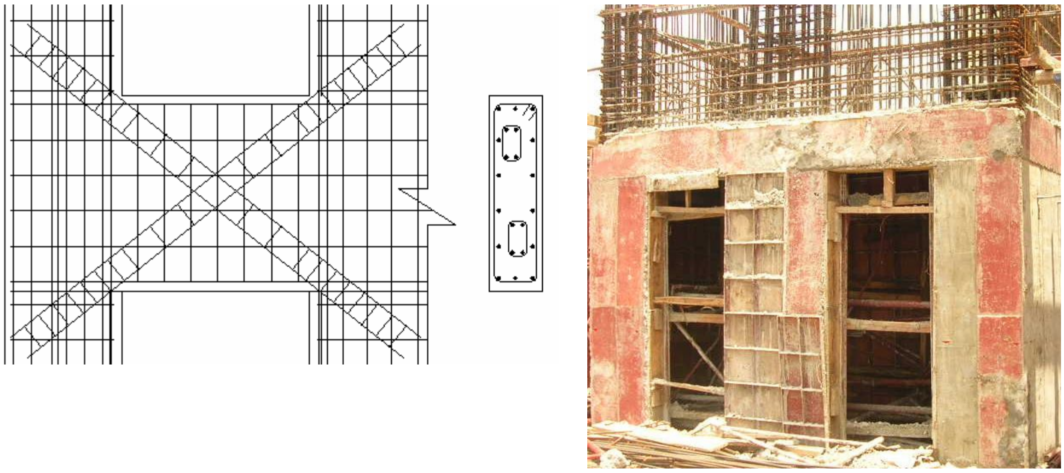


a)



b)

Figure 2.13-Plan view of core wall and coupling beams (a)
and section of coupling beams with diagonal reinforcement rods (b)



a) b)

Figure 2-14. Coupling beams in drawing (a) and actual site (b): Diagonal bars cage becomes reinforcement rods

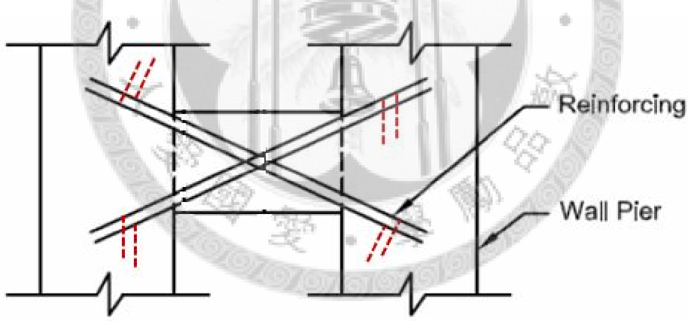


Figure 2.15-Anchoring bend of diagonal bars in coupling beams (dashed line)

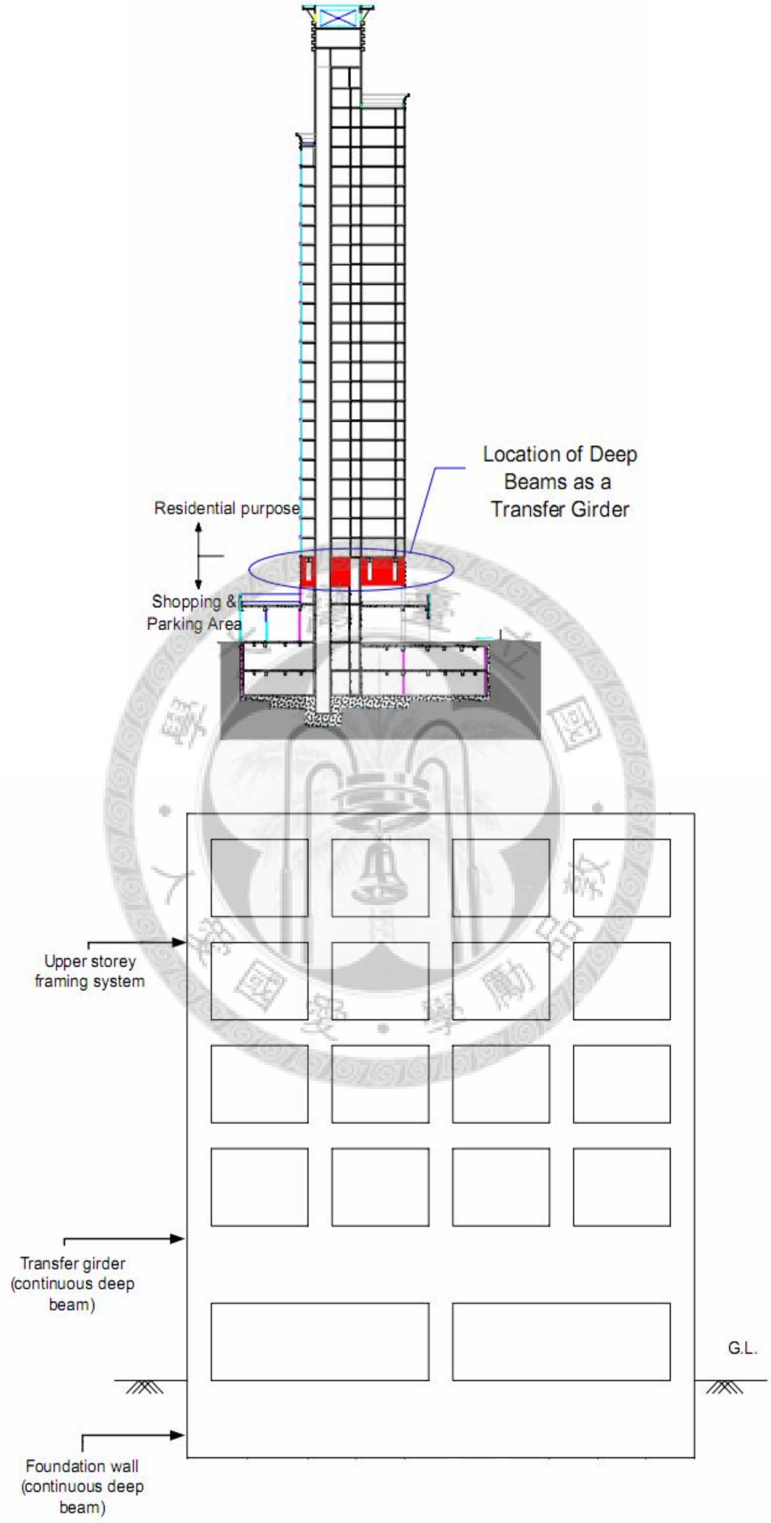
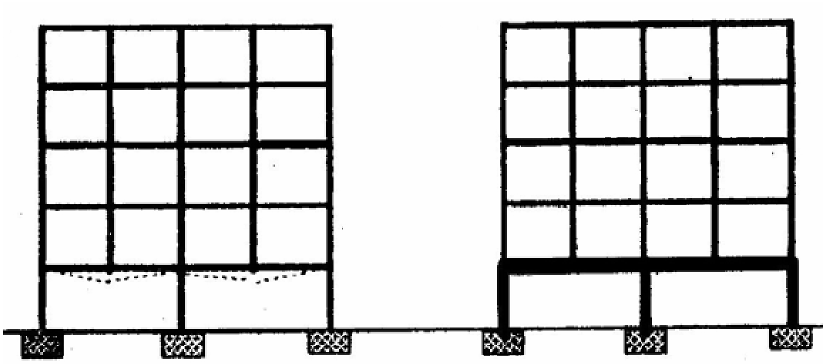


Figure 2.16-Example of continuous deep beams (Elevated view)



a) Solution should not to use **b) Measure to overcome**

Figure 2.17-Guidance of detail for monolithic reinforced concrete frame in TCXD 198:1997 [7]

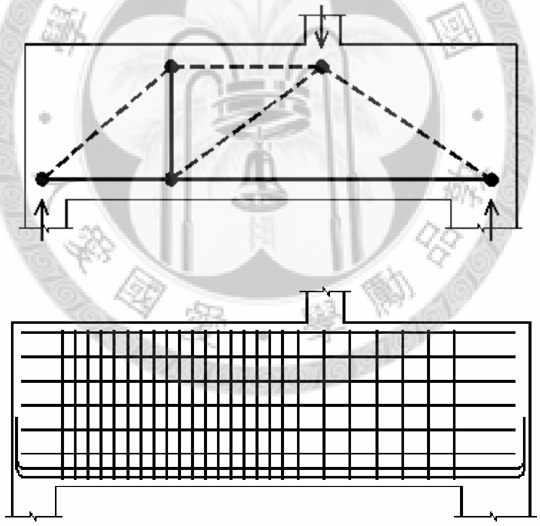


Figure 2.18-Using strut-and-tie models in single span deep beams

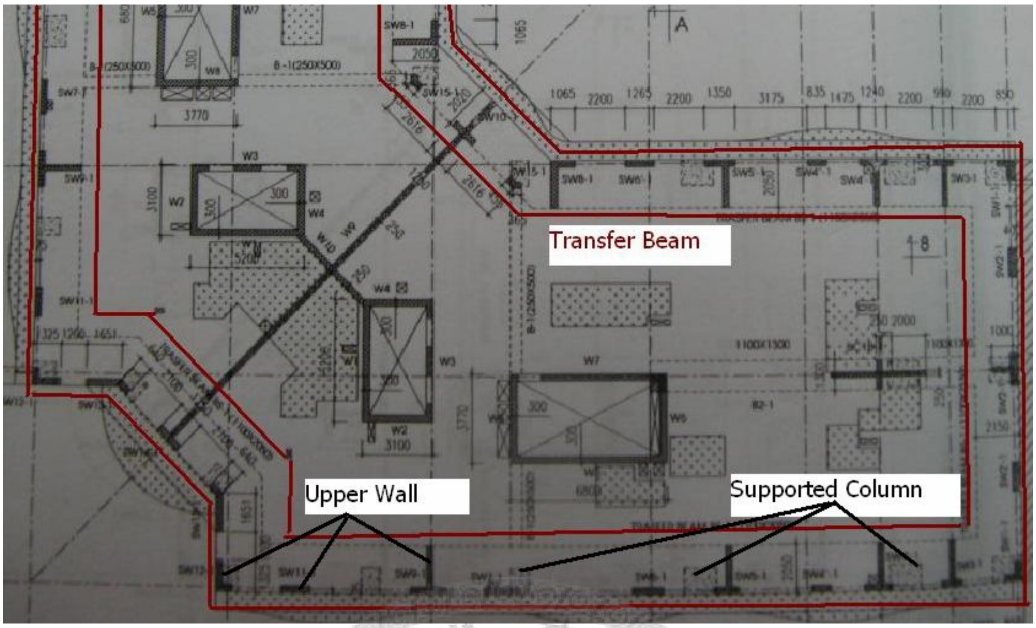


Figure 2.19-Structural plan with transfer beams

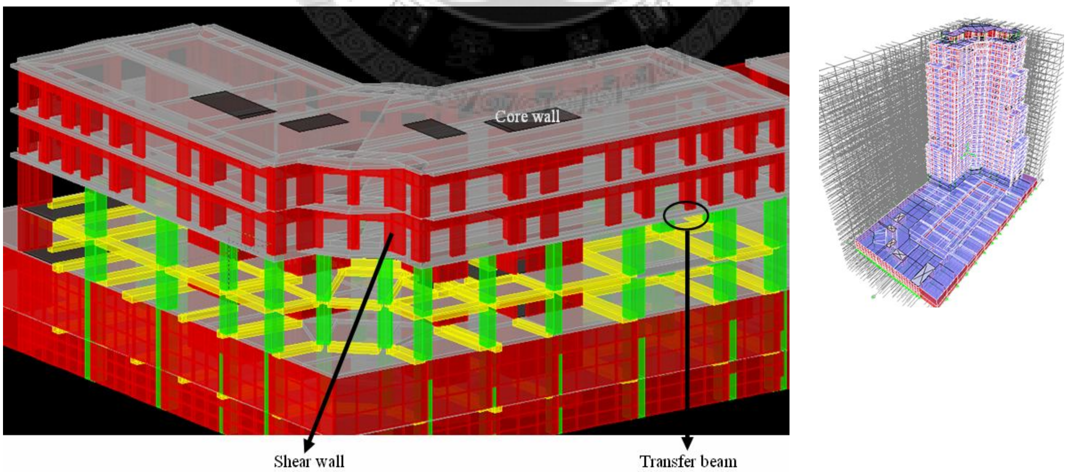


Figure 2.20-3D view of analysis model with transfer beams in ETABS

Figure 2.21-Photos of transfer beams during construction

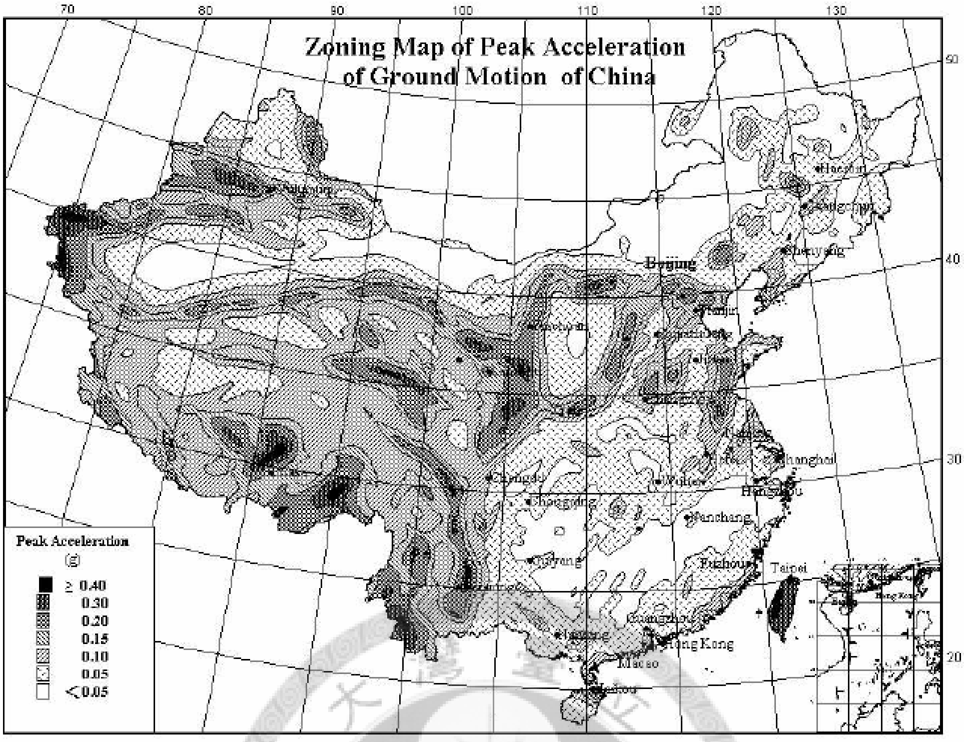


Figure 2.22-Seismic zonation map of peak ground acceleration of China [21]

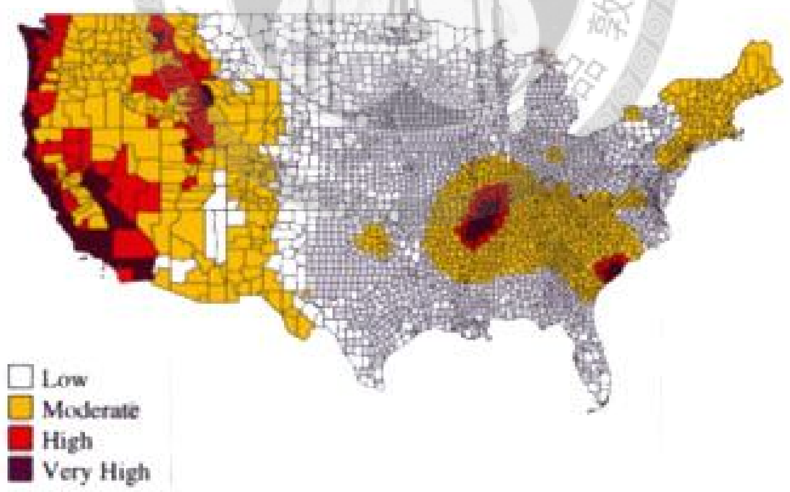


Figure 2.23-Seismicity of the United States [22]

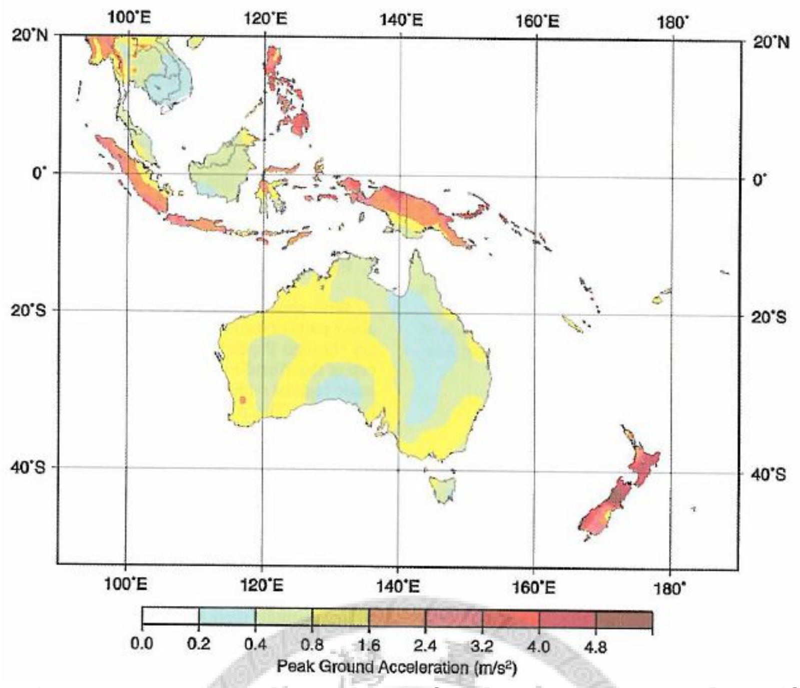


Figure 2.24-Seismic hazard map for the Australia, South Pacific, and Southeast Asia region [23]

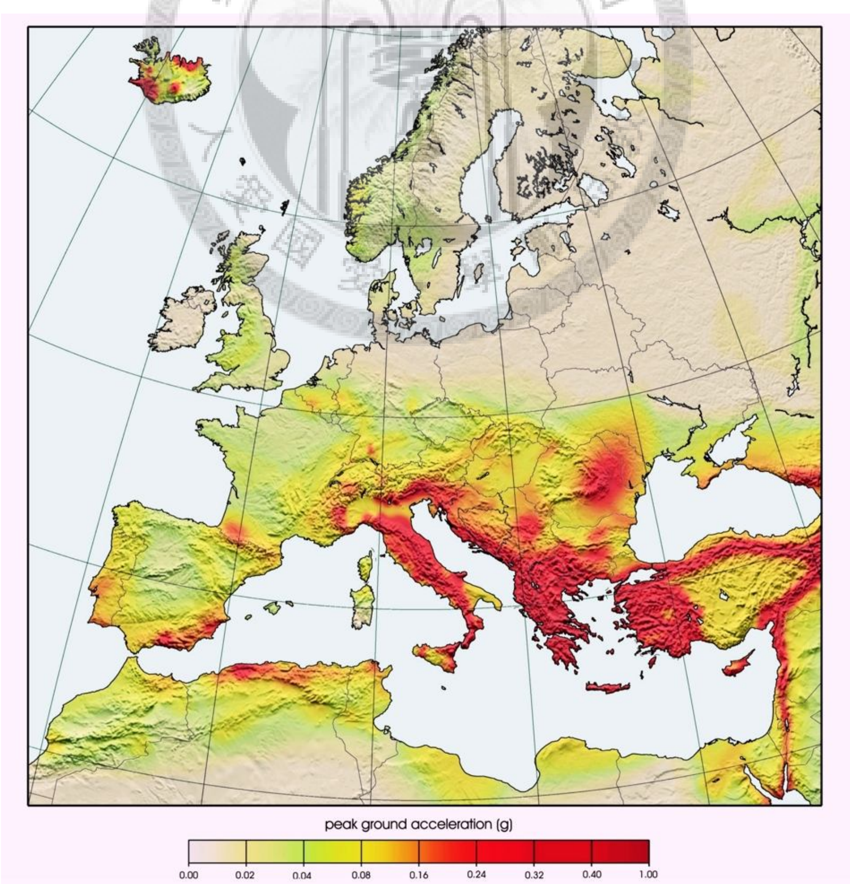
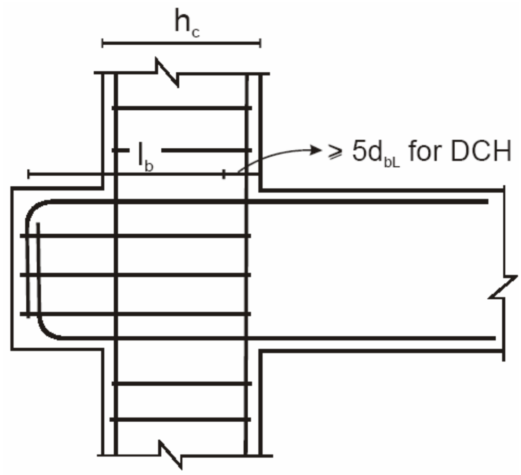
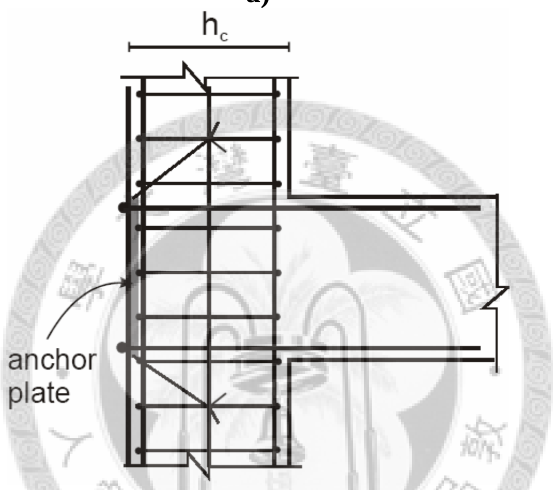


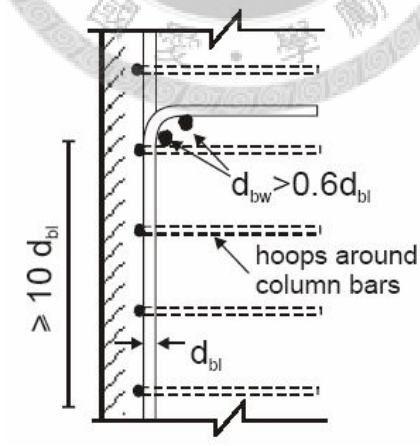
Figure 2.25-European-Mediterranean seismic hazard map for the peak ground acceleration [24]



a)



b)



c)

Figure 3.1-Additional measures for anchorage in exterior beam-column joints (Figure 5.13 in EN 1998-1:2004 [4])

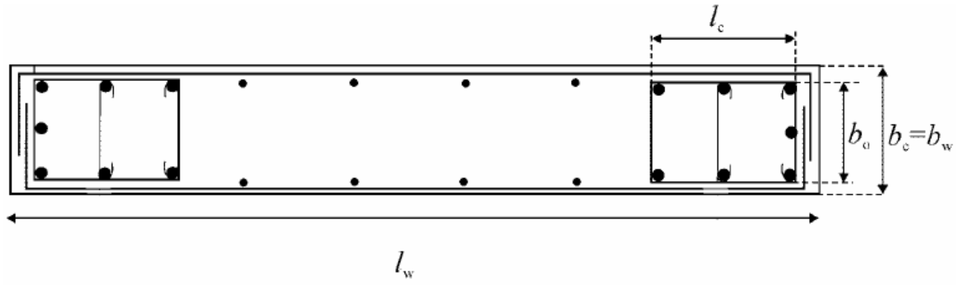
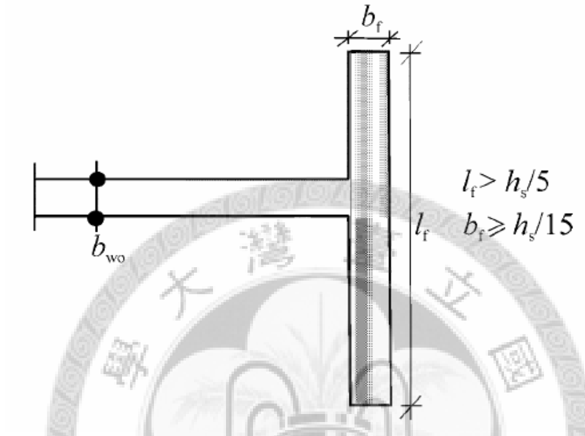
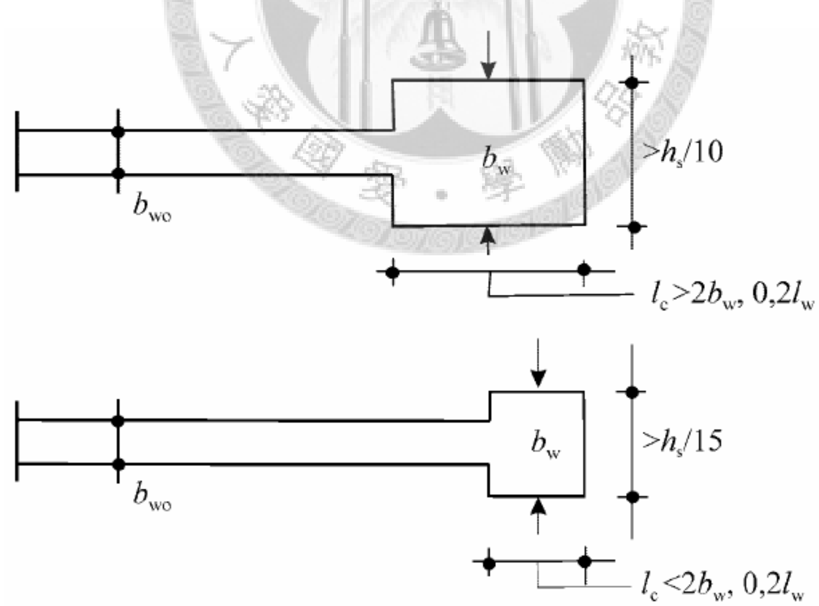


Figure 3.2-Confined boundary element of free-edge wall end
(Figure 5.8 in EN 1998-1:2004 [4])

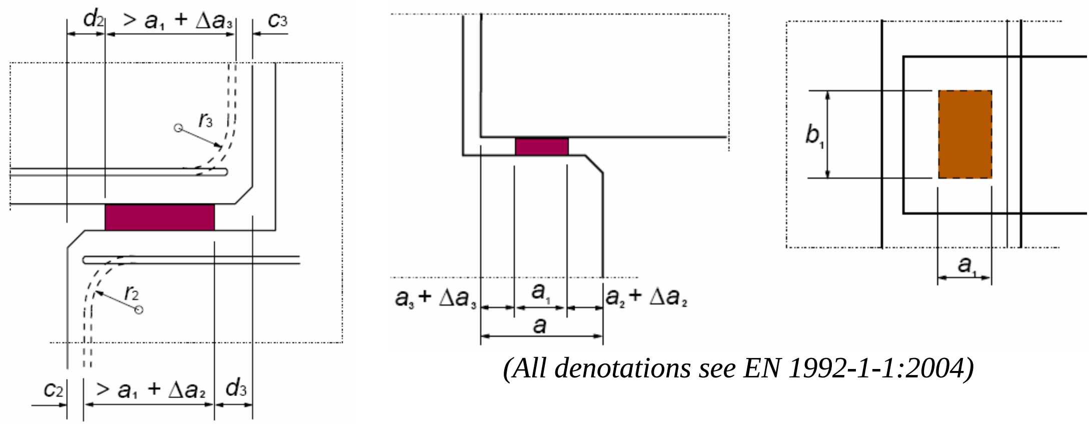


a)



b)

Figure 3.3-Detailing of confined boundary elements:
a) Confined boundary element not needed at wall end with a large transverse flange;
b) Minimum thickness of confined boundary elements
(Figure 5.9, 5.10 in EN 1998-1:2004 [4])



a) **b)**

Figure 3.4-Example of bearing: a) Detailing of reinforcement in support; b) Bearing with definitions (Figure 10.5 and 10.6 in EN 1992-1-1:2004 [9])

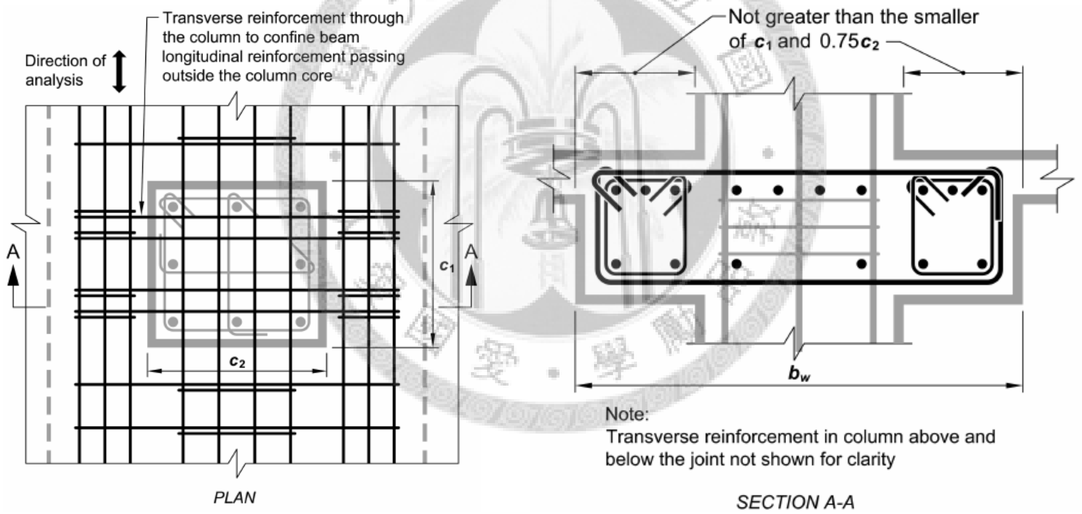


Figure 3.5-Maximum effective width of wide beam and required transverse reinforcement (Figure R21.5.1 in ACI 318-08 [2])

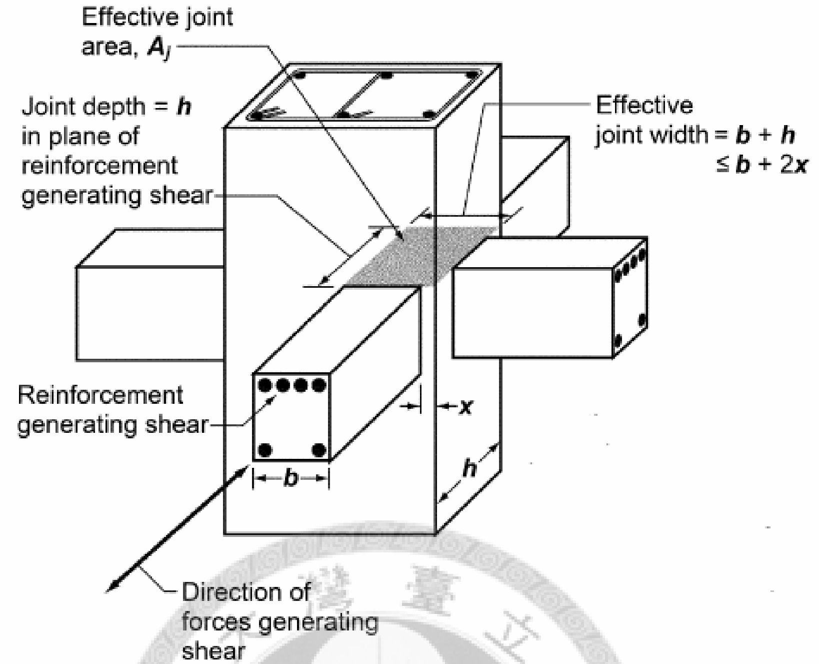


Figure 3.6-Effective joint area
(Figure R21.7.4 in ACI 318-08 [2])

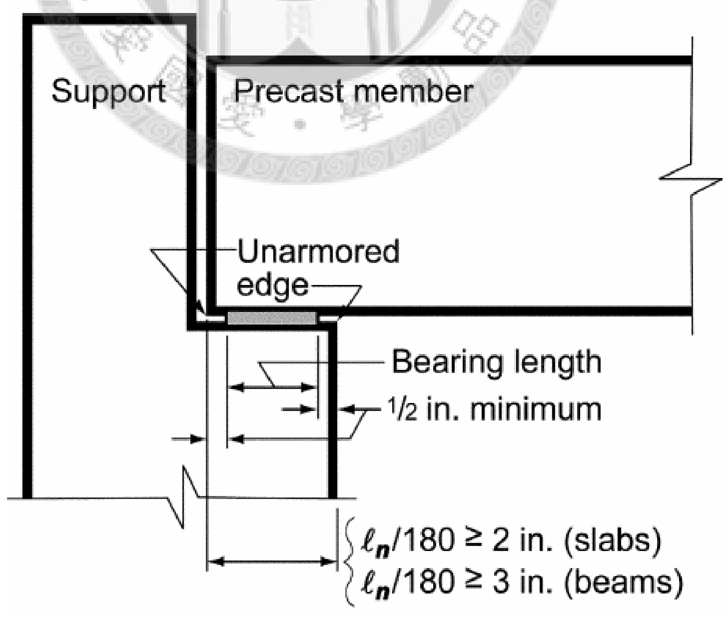
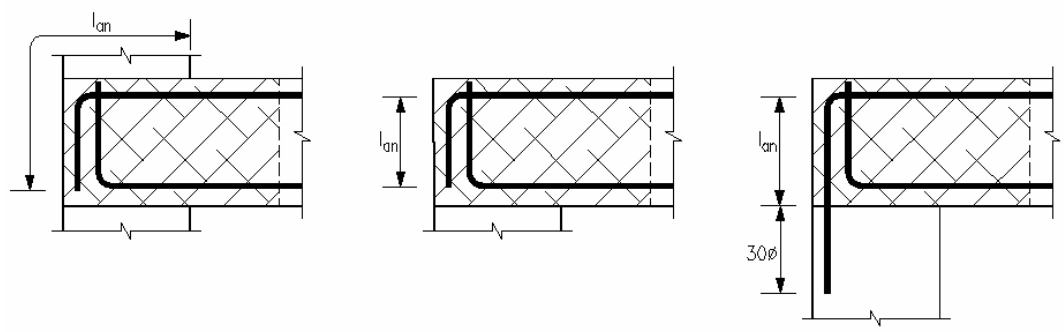
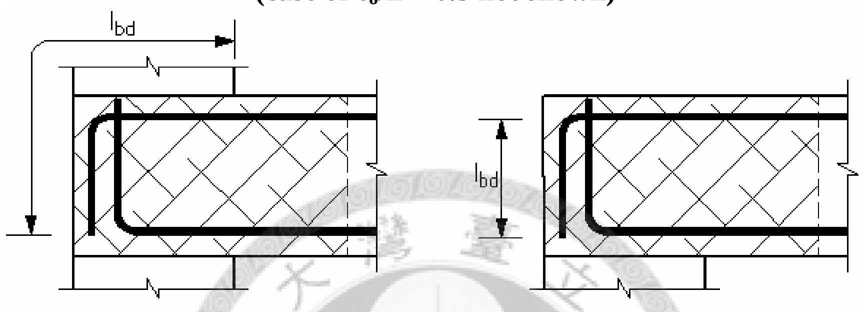


Figure 3.7-Bearing length on support
(Figure R16.6.2 in ACI 318-08 [2])

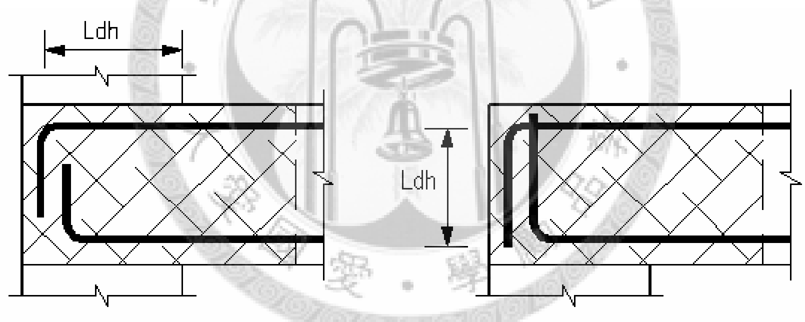


$e_o/h \geq 0.25$ *$0.25 < e_o/h \leq 0.5$*

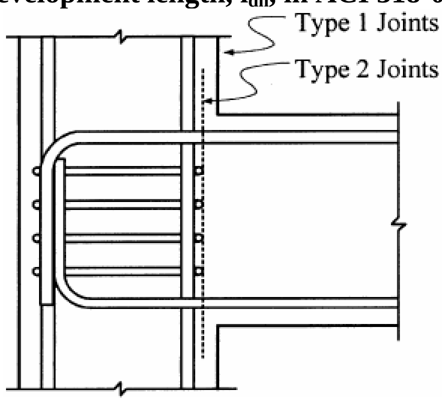
a) Development length, l_{an}, in TCXDVN 356:2005 [8]
(case of $e_o/h > 0.5$ not shown)



b) Development length, l_{bd}, in EN 1992-1-1:2004 [10]

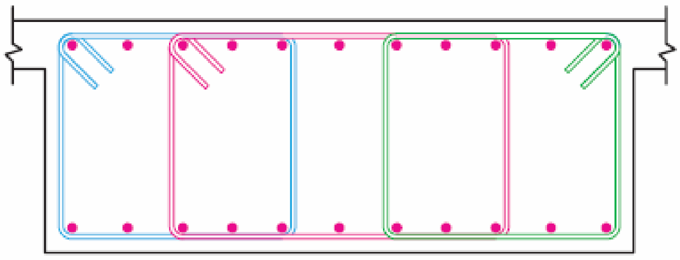


c) Development length, l_{dh}, in ACI 318-08 [2]

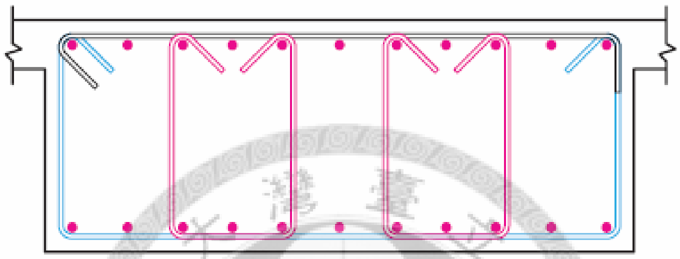


d) Critical section for development length (Figure 4.8, ACI 352R-02 [29])

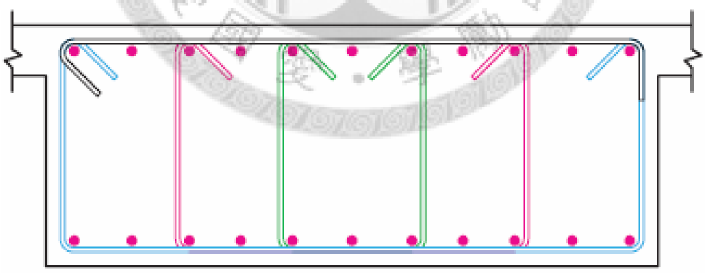
Figure 3.8-Different development length in codes
(Elevated sections of wide beam-column exterior (left) and roof-corner joints (right))



a) Beam stirrup configuration with three closed stirrups distributed across the beam width

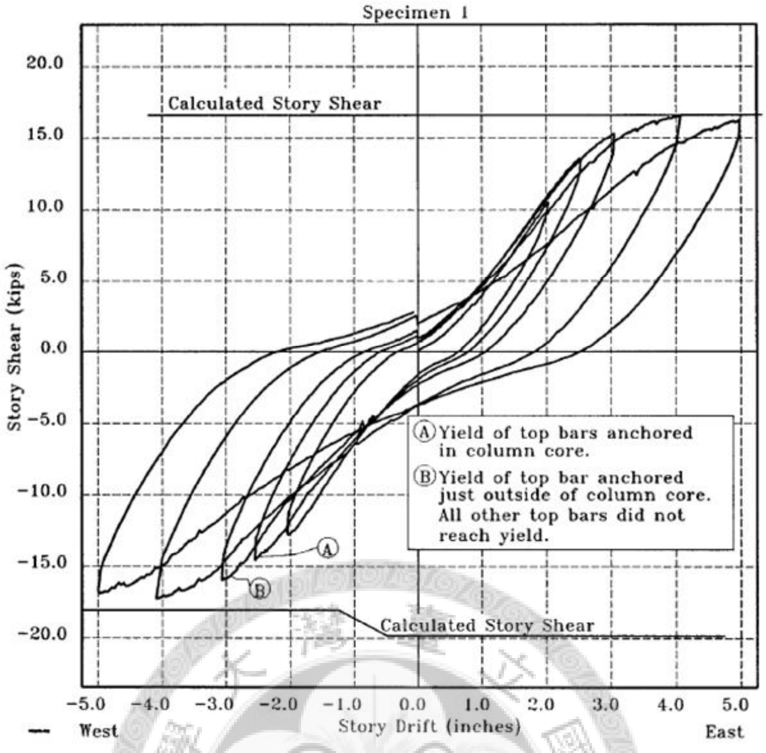


b) An alternate configuration consisting of a single U-stirrup (with 135-degree hooks) across the net width of the beam, two identical U-stirrups (each with 135-degree hooks) distributed across the beam interior, and a stirrup cap

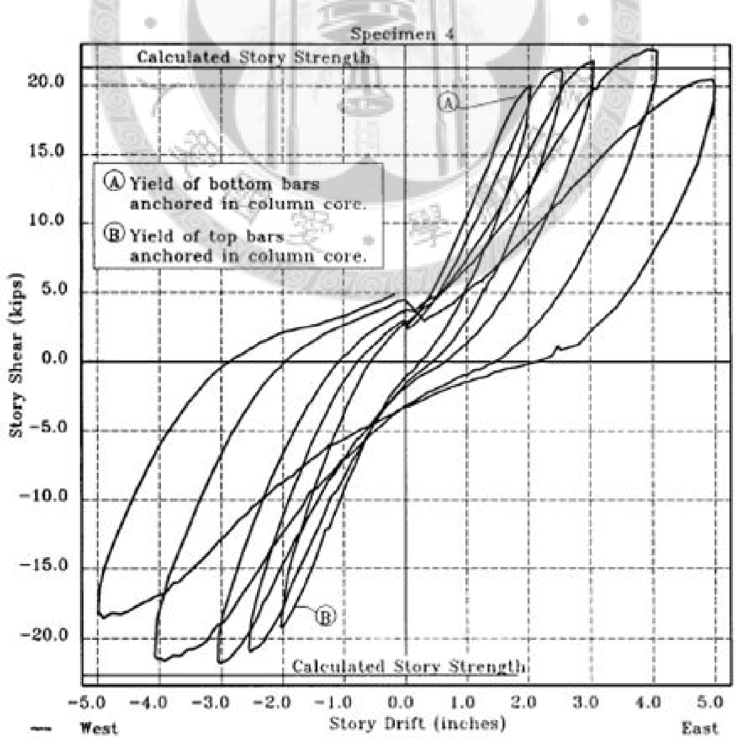


c) A second alternate configuration consisting of a single U-stirrup across the net width of the beam, two smaller-width U-stirrups nested in the beam interior, and a stirrup cap

Figure 3.9-Stirrup configurations in wide beam [43]

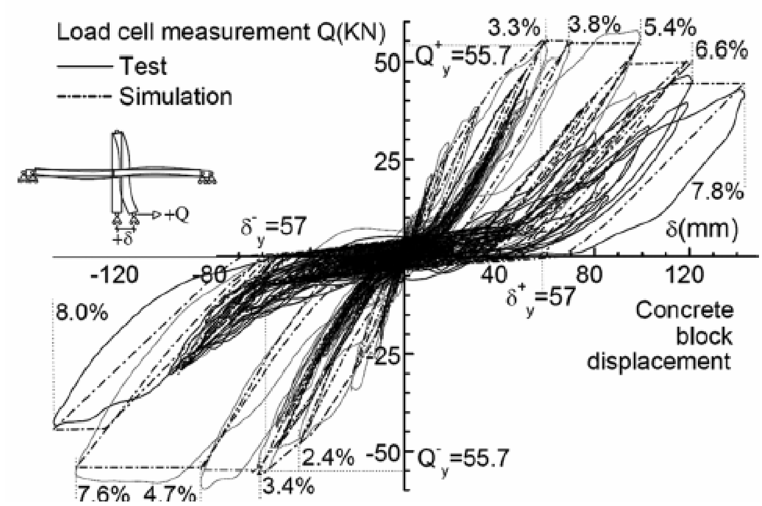


a) Specimen 1-Exterior joint test by Gentry and Wight [16]

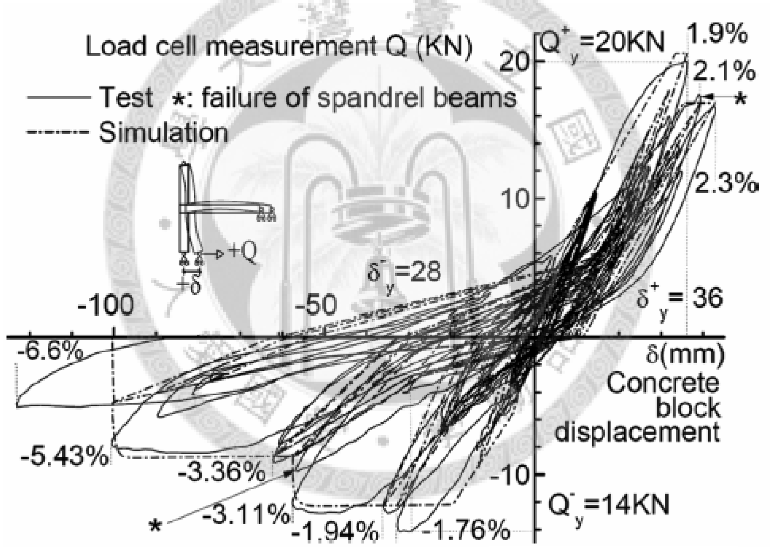


b) Specimen 4-Exterior joint test by Gentry and Wight [16]

Figure 3.10-Lateral load-displacement hysteretic loop

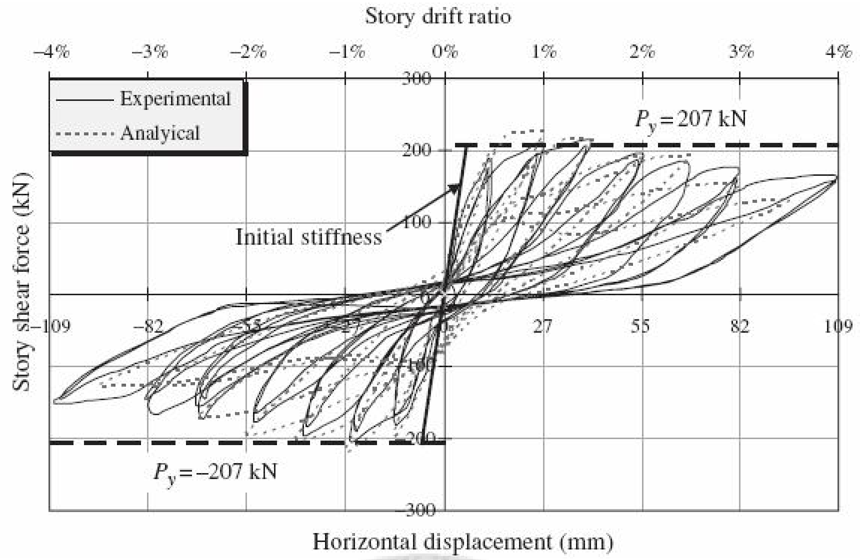


c) Specimen IWB-Interior joint test by Benavent-Climent [26]

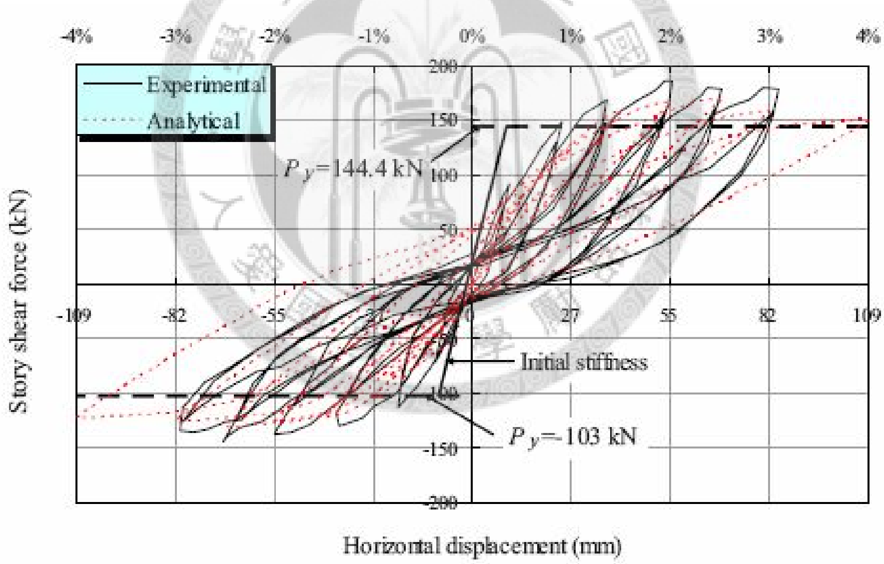


d) Specimen EWB-Exterior joint test by Benavent-Climent [26]

Figure 3.10-Lateral load-displacement hysteretic loop

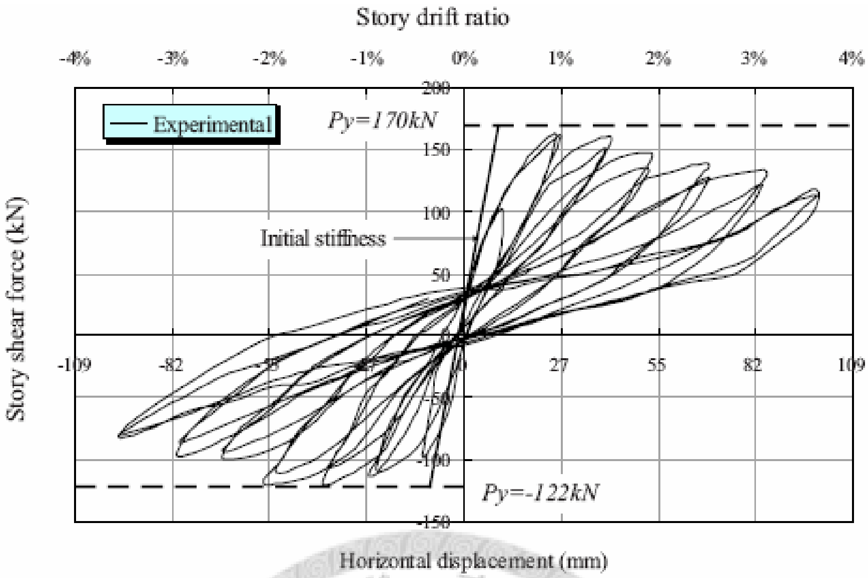


e) Specimen IWB2-Interior joint test by Kulkarni and Li [36]

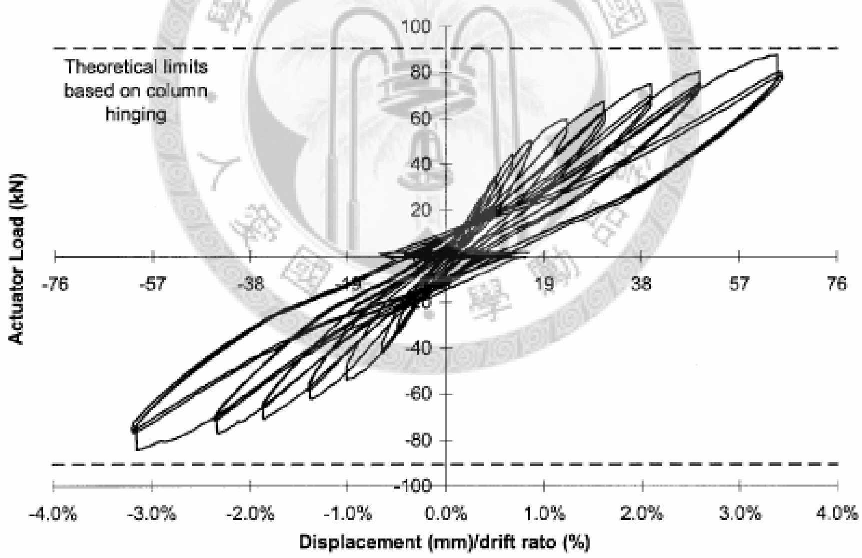


f) Specimen EWB2-Exterior joint test by Li and Kulkarni [37]

Figure 3.10-Lateral load-displacement hysteretic loop

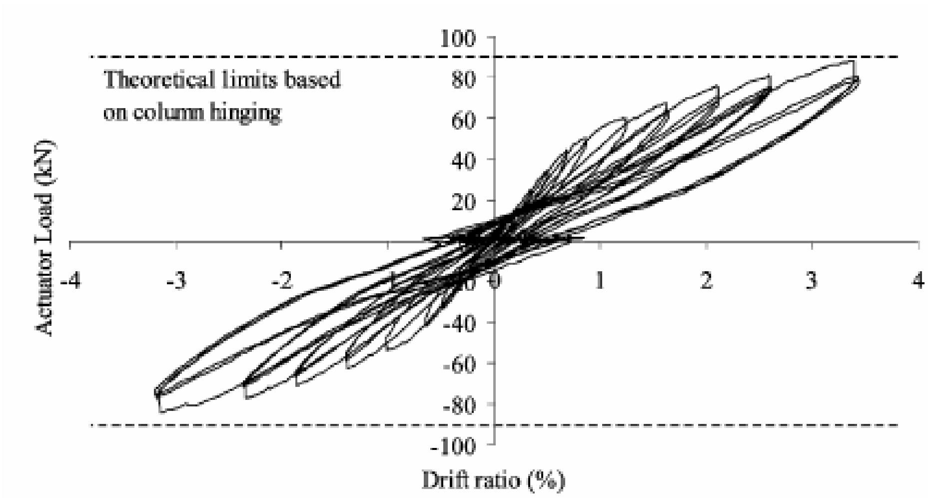


g) Specimen EWB3-Exterior joint test by Li and Kulkarni [37]

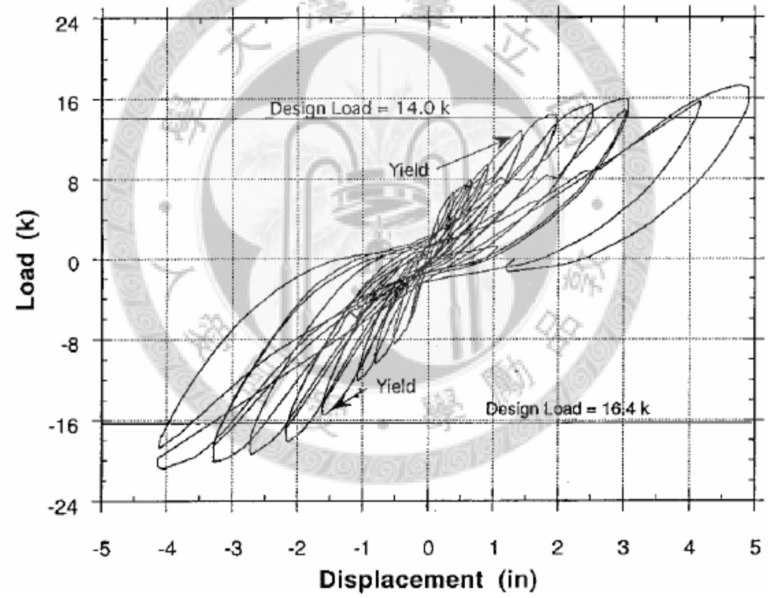


h) Specimen No.2-Interior joint test by Stehle et al. [40]

Figure 3.10-Lateral load-displacement hysteretic loop



i) Specimen WBB-I2-Interior joint test by Siah et al. [41]



j) Specimen EWB-1-Exterior joint test by LaFave and Wight [42]

Figure 3.10-Lateral load-displacement hysteretic loop

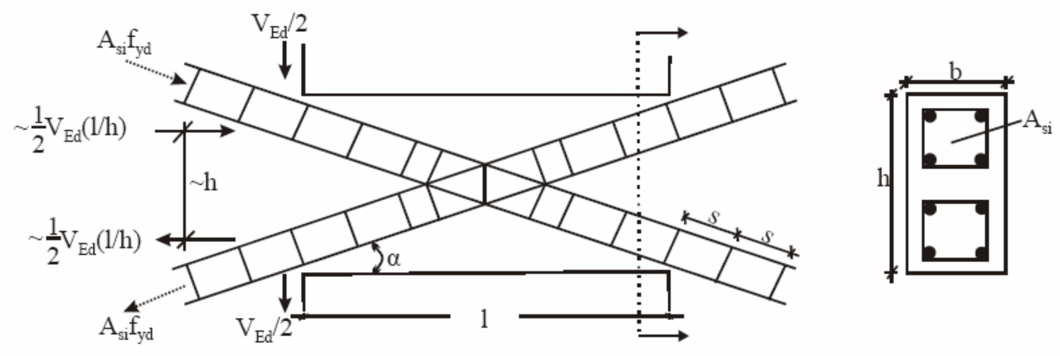


Figure 4.1-Confinement of individual diagonals in coupling beams with diagonally oriented reinforcement (Figure 5.12 in EN 1998-1:2004 [4])

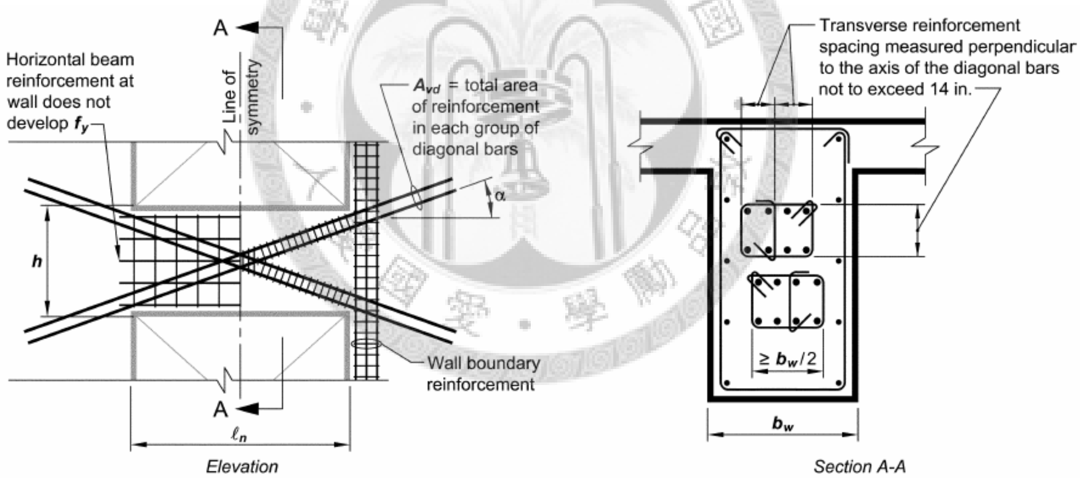


Figure 4.2-Confinement of individual diagonals in coupling beams with diagonally oriented reinforcement (Figure R21.9.7.a in ACI 318-08 [2])

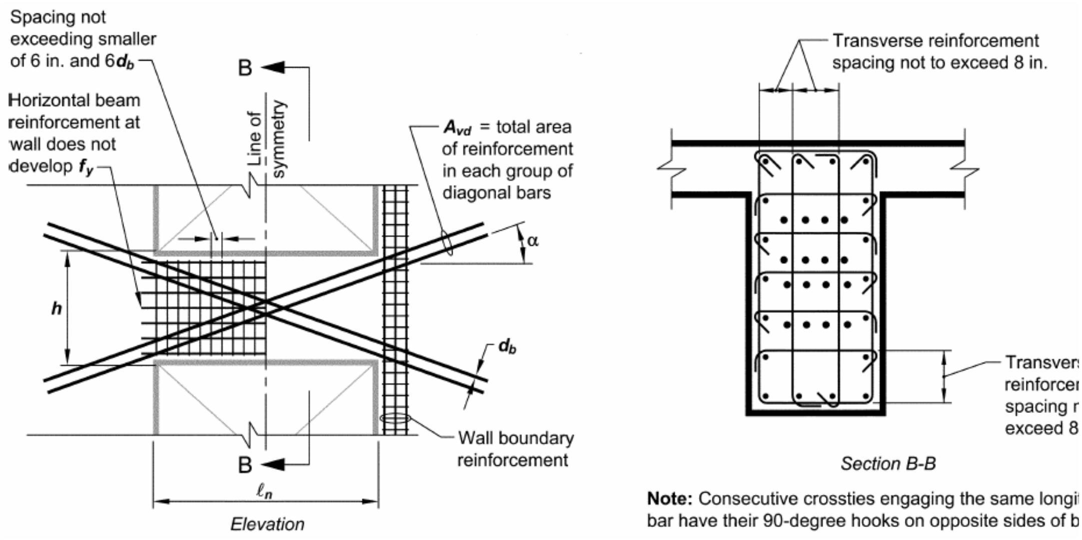
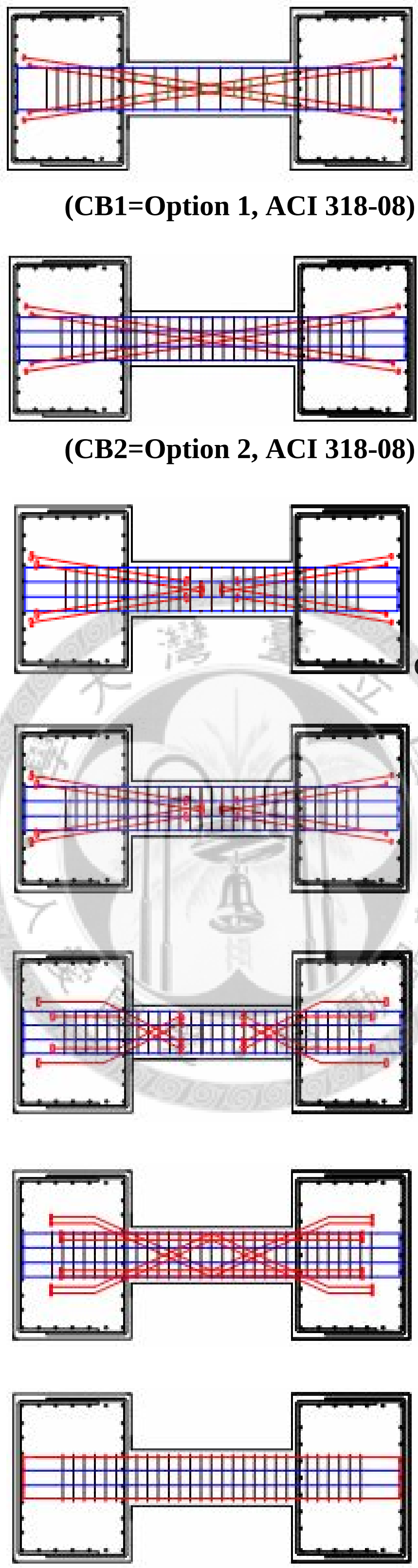
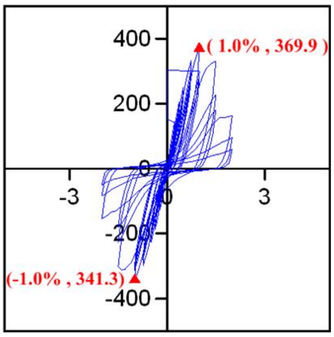


Figure 4.3-Full confinement of diagonally reinforced concrete beam section in coupling beams with diagonally oriented reinforcement (Figure R21.9.7.b in ACI 318-08 [2])



CB1
(CB1=Option 1, ACI 318-08)
CB2
(CB2=Option 2, ACI 318-08)
CB3
CB4
CB5
CB6
CB7

Figure 4.4-Coupling beams: 7 specimens for experimental test (l/h=3) [46]



Hysteretic loop

Drift ratio at peak strength: 1%

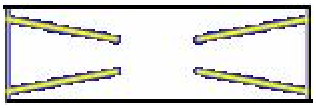
V_u = 369.9 (kN)

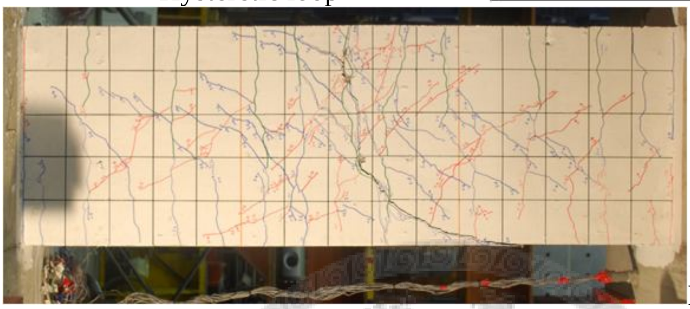
V_n = 822.9 kN > V_u

$V_{n(mid\text{-}span)}$ = 419 kN > V_u

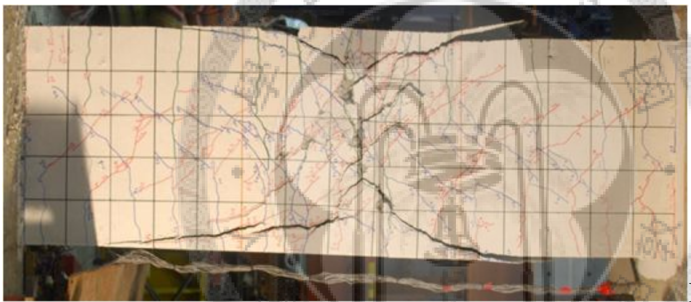
V_{sf} = 295.3 kN < V_u

(V_{sf}: Shear friction)

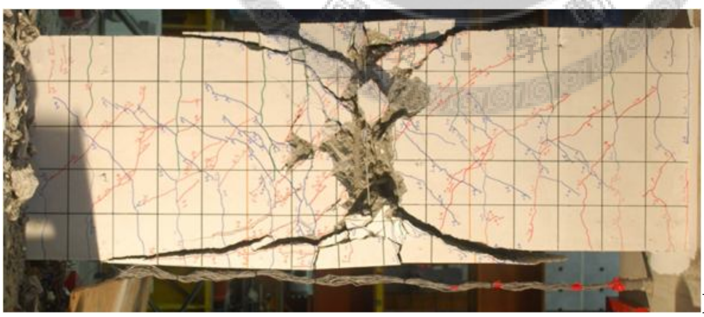




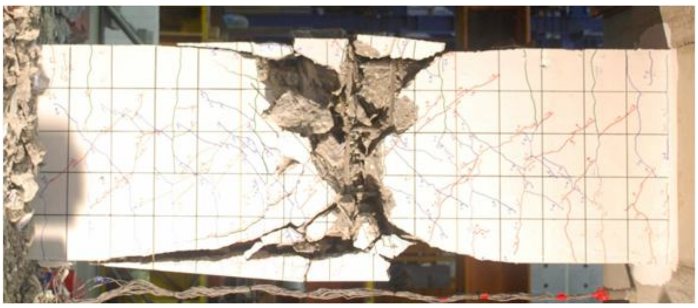
Drift ratio = +1%



Drift ratio = -1.5%



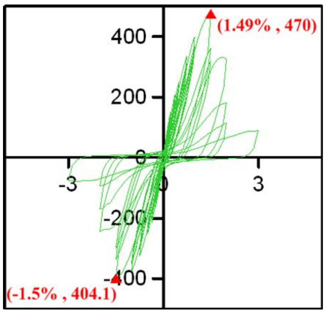
Drift ratio = +2.0%



Drift ratio = Final

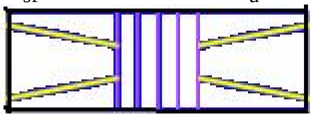
a) Specimen CB-3

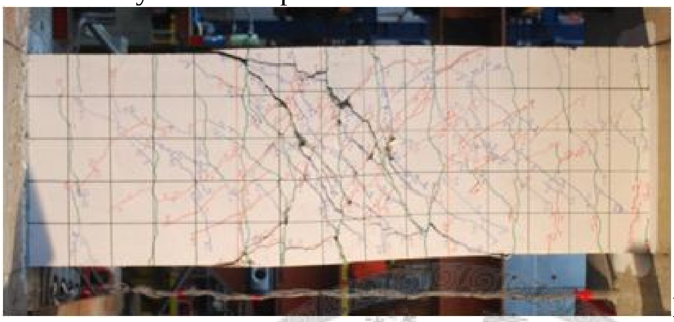
Figure 4.5-Result of experimental tests for coupling beams [46]

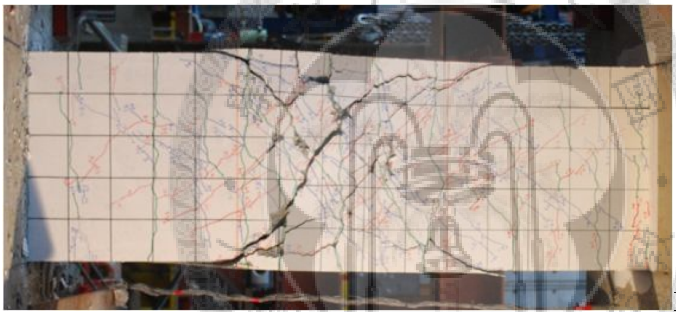


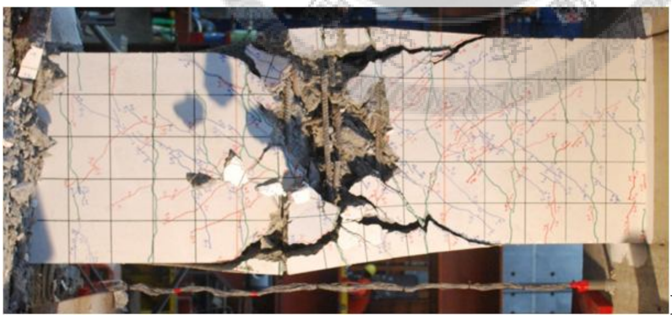
Hysteretic loop

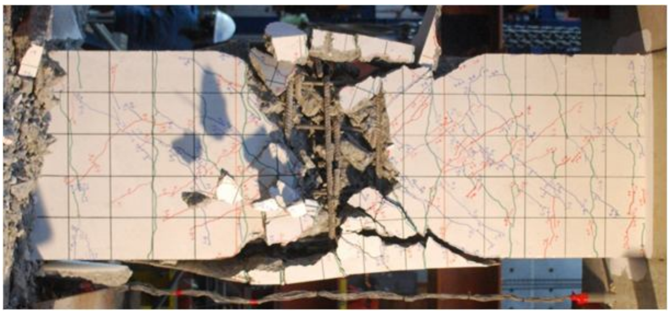
Drift ratio at peak strength: 1.49%
V_u = 470 (kN)
V_n = 822.9 kN > V_u
$V_{n(mid\text{-}span)}$ = 669 kN > V_u
V_{sf} = 295.3 kN < V_u




Drift ratio = +2.0%

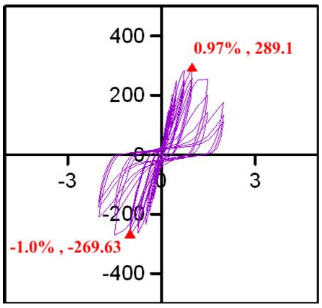

Drift ratio = -2.0%


Drift ratio = +3.0%


Drift ratio = Final

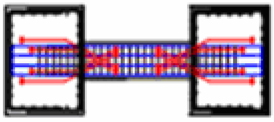
b) Specimen CB-4

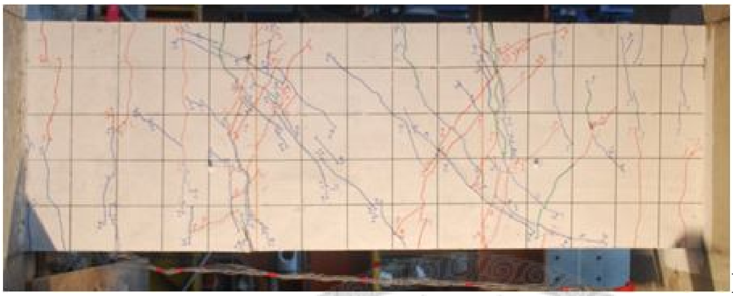
Figure 4.5-Result of experimental tests for coupling beams [46]

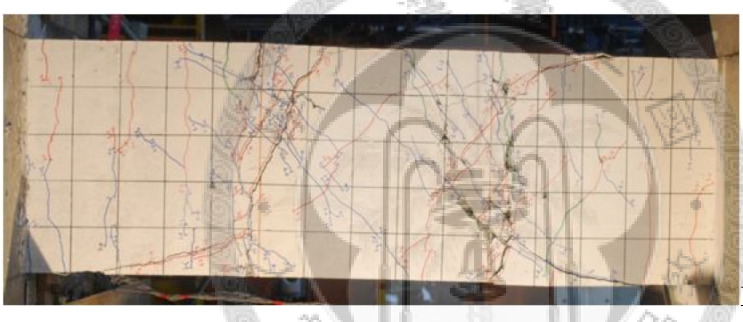


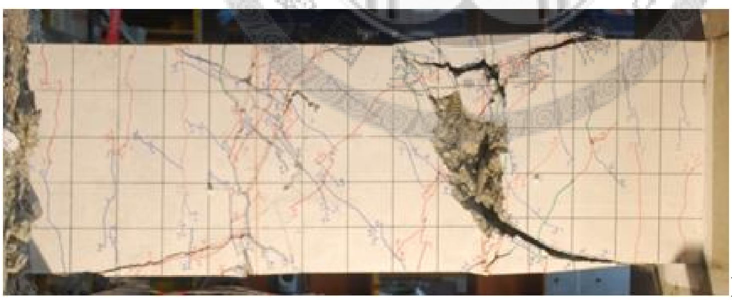
Hysteretic loop

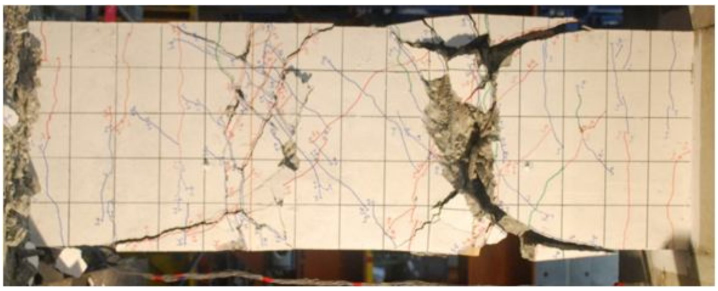
Drift ratio at peak strength: 0.97%
V_u = 289.1 (kN)
V_n = 822.9 kN > V_u
$V_{n(mid\text{-}span)}$ = 669 kN > V_u
V_{sf} = 295.3 kN > V_u




Drift ratio = +1%

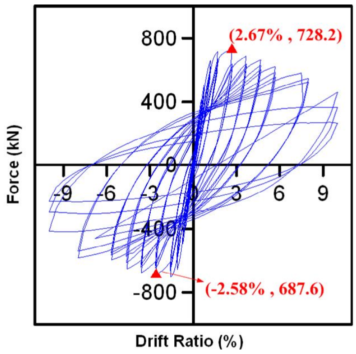

Drift ratio = -1.5%


Drift ratio = +2%


Drift ratio = Final

c) Specimen CB-5

Figure 4.5-Result of experimental tests for coupling beams [46]

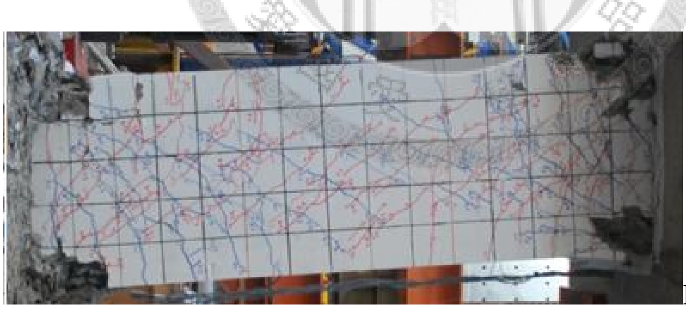


Hysteretic loop

Drift ratio at peak strength: 2.67%
V_u = 728.2 (kN)
V_n = 822.9 kN > V_u

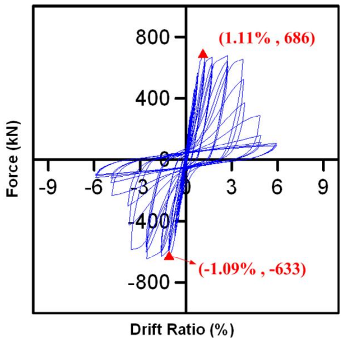

Drift ratio = +2%


Drift ratio = +3%


Drift ratio = +6%

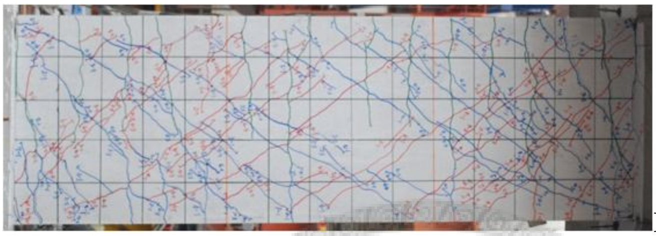

Drift ratio = Final

d) Specimen CB-2 (Option 2, ACI 318-08)
Figure 4.5-Result of experimental tests for coupling beams [46]

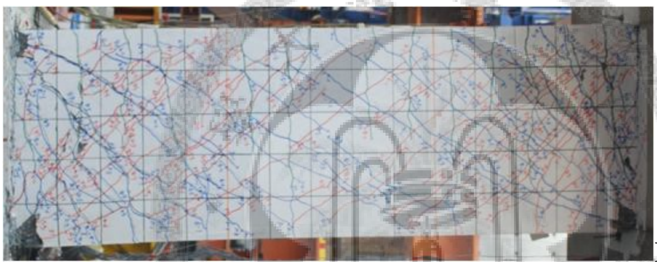


Hysteretic loop

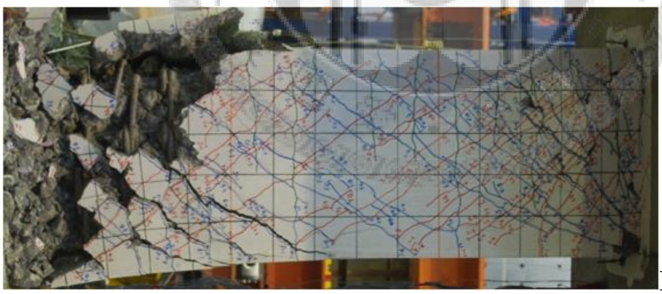
Drift ratio at peak strength: 1.11%
V_u = 686 (kN)
V_n = 804.3 kN > V_u



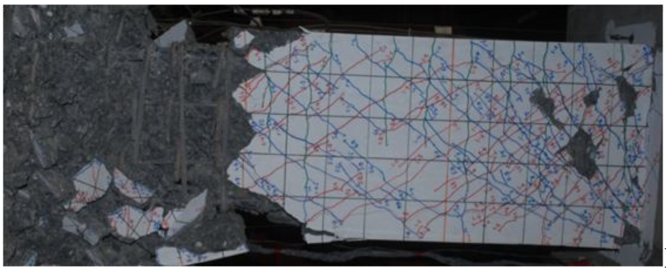
Drift ratio = +2%



Drift ratio = +4%



Drift ratio = +5%



Drift ratio = Final

e) Specimen CB-7 (Without diagonal bars)

Figure 4.5-Result of experimental tests for coupling beams [46]

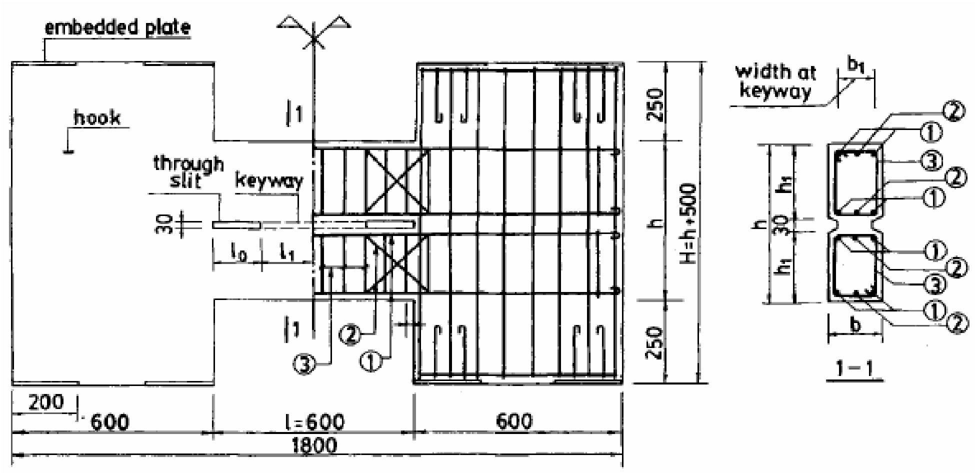
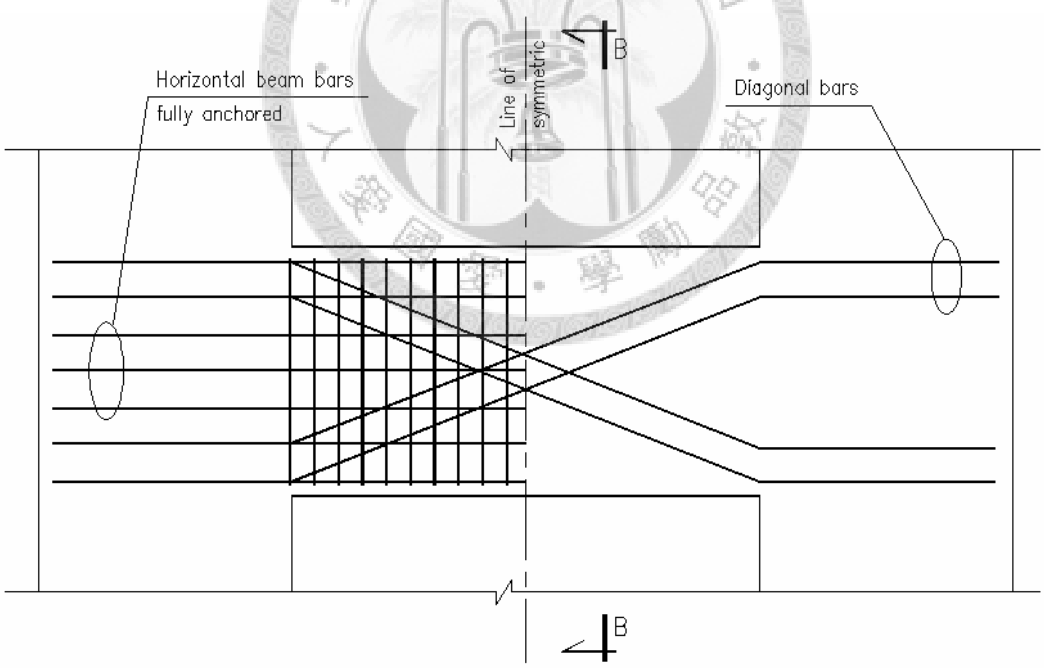


Figure 4.6-Coupling beams with slits and keyways [45]



(Section B-B: See Section B-B in Figure 4.3)

Figure 4.7-Proposed solution: Second option in ACI 318-08 [2] (Figure R21.9.7.b) and Lequesne et al. [47]

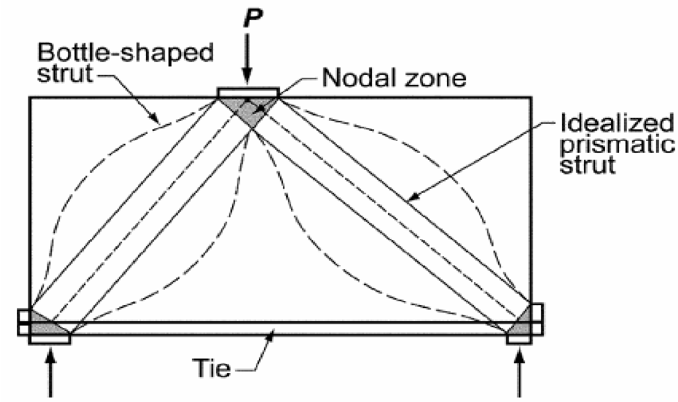
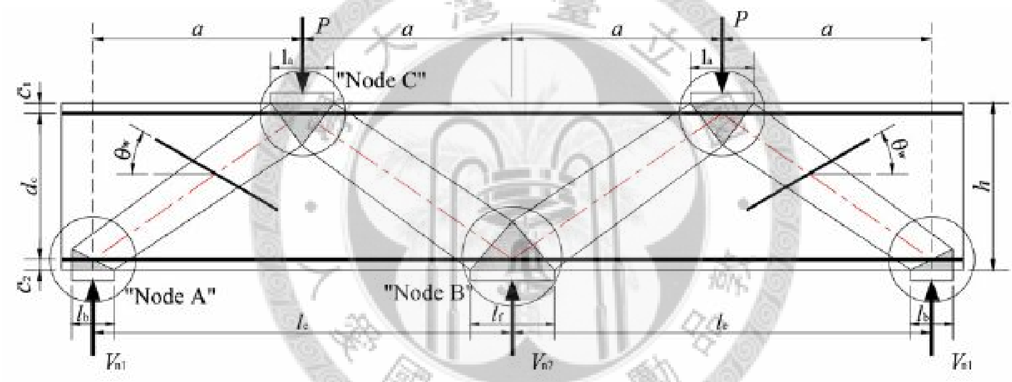
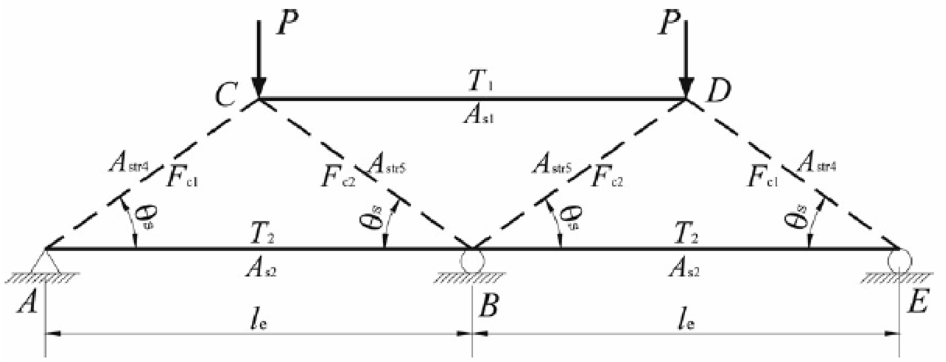


Figure 5.1-Description for strut-and-tie models
(case of single-span deep beams loaded with a concentrated load)
(Figure RA.1.3 in ACI 318-08 [2])



g: a) Detailing of reinforcement in support; b) Bearing with definitions (Figure



b) Truss model

Figure 5.2-Two span continuous deep beams [61]

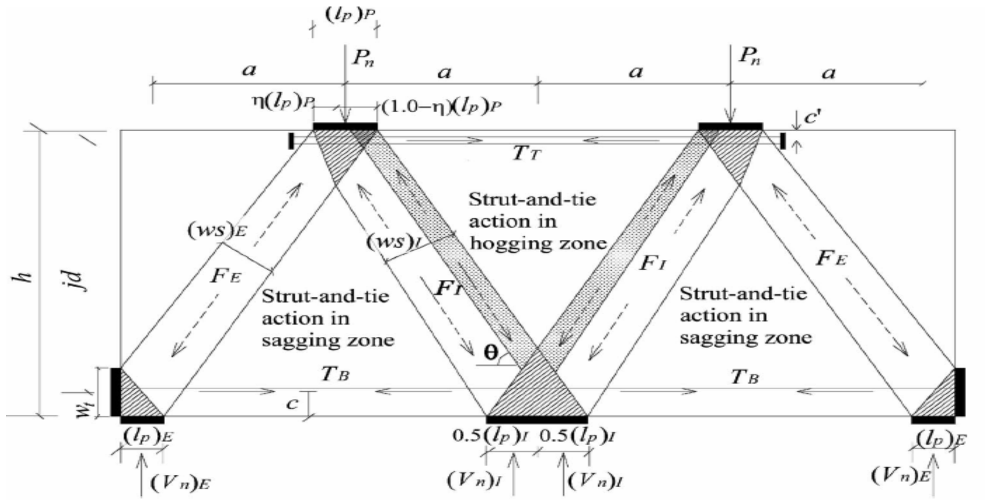
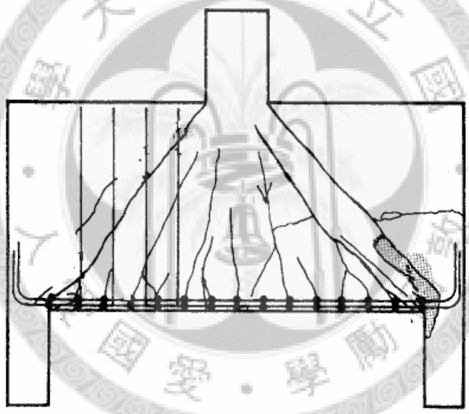
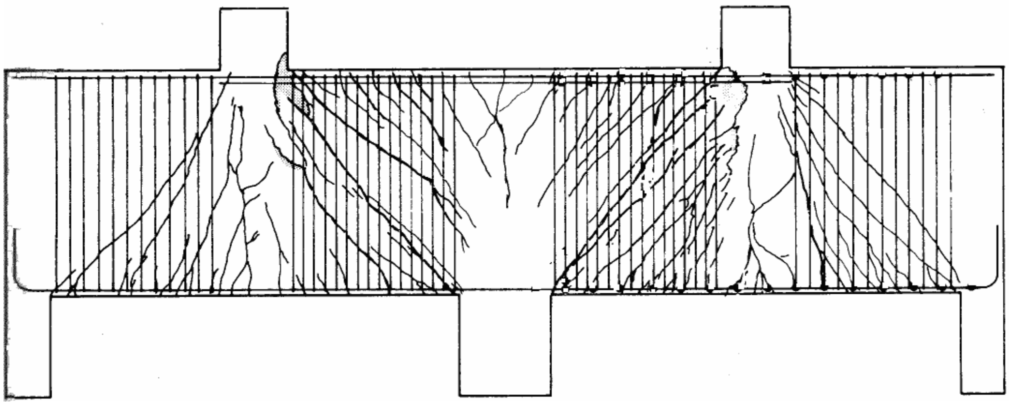


Figure 5.3-Schematic STM for continuous deep beams based on ACI 318-05 [64]



a) Simple deep beams



b) Two span deep beams

Figure 5.4-Crack pattern at failure [62]

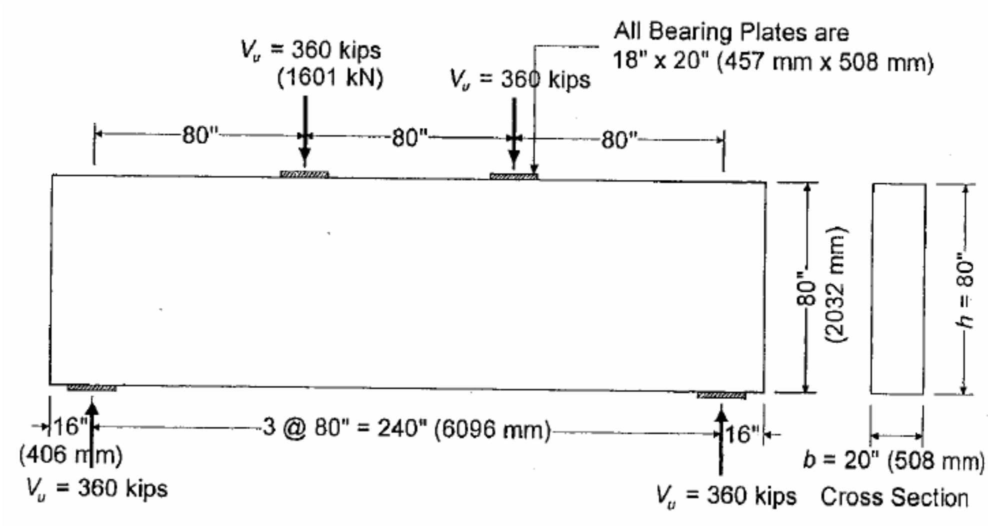
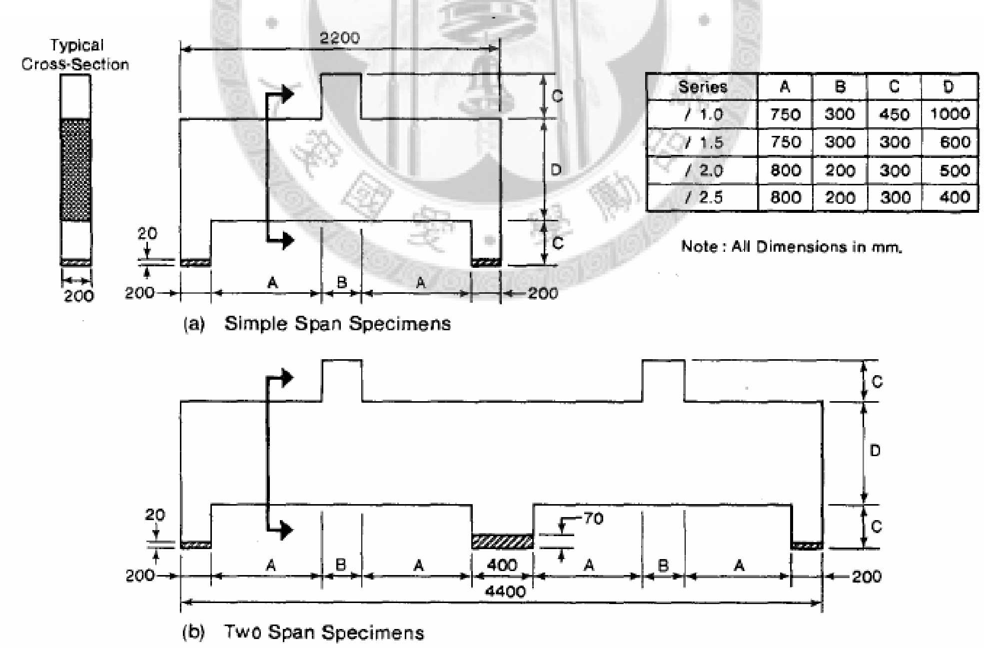


Figure 5.5-Span deep beams with bearing plates at loading and support locations [69]



Series	A	B	C	D
/ 1.0	750	300	450	1000
/ 1.5	750	300	300	600
/ 2.0	800	200	300	500
/ 2.5	800	200	300	400

Figure 5.6-Specimens of simple span and two span deep beams with column stubs [62]

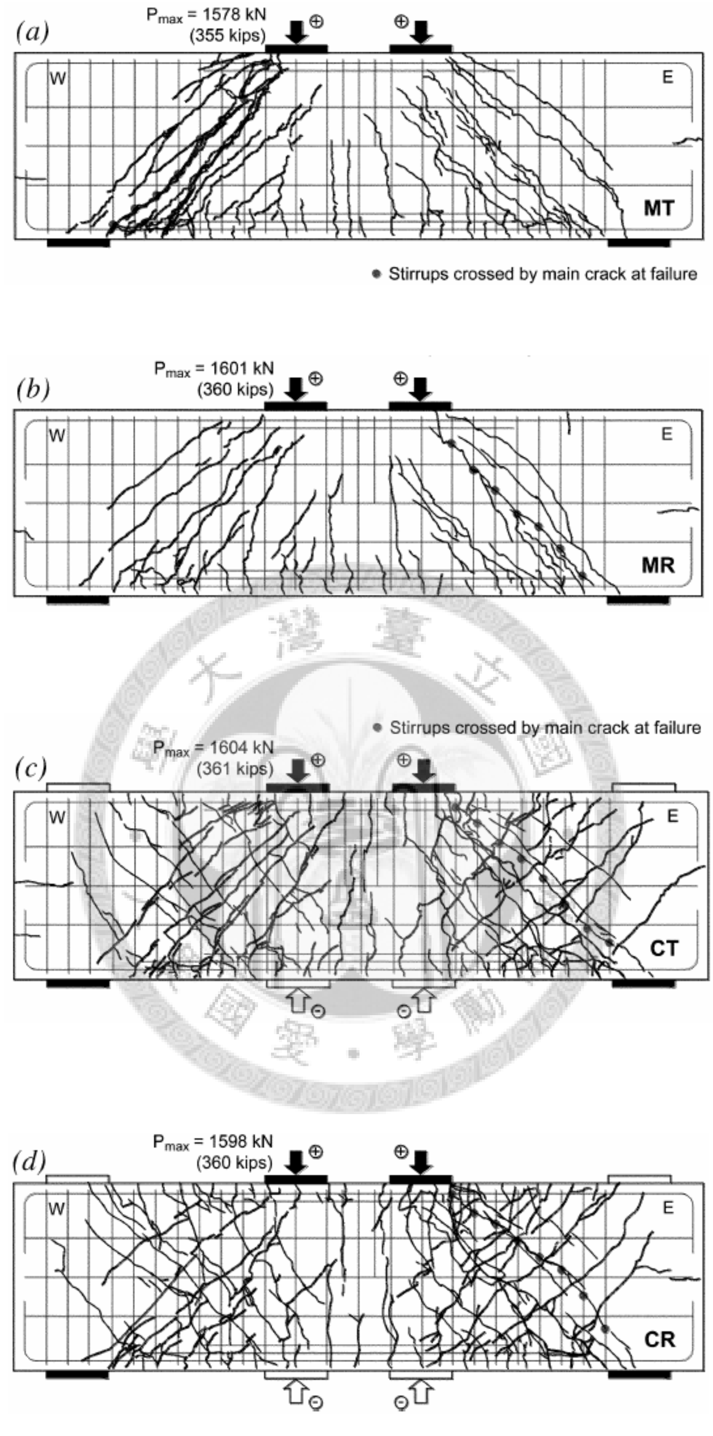


Figure 5.7-Final crack patterns in 4 specimens [79]

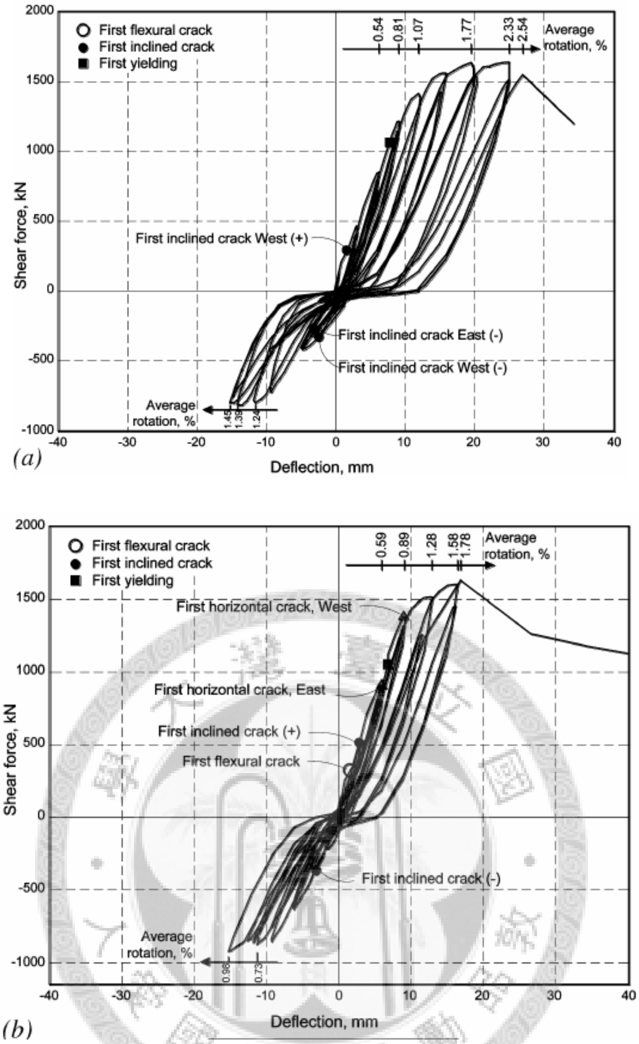


Figure 5.8-Hysteretic curves: a) Beam CT; b) Beam CR [79]

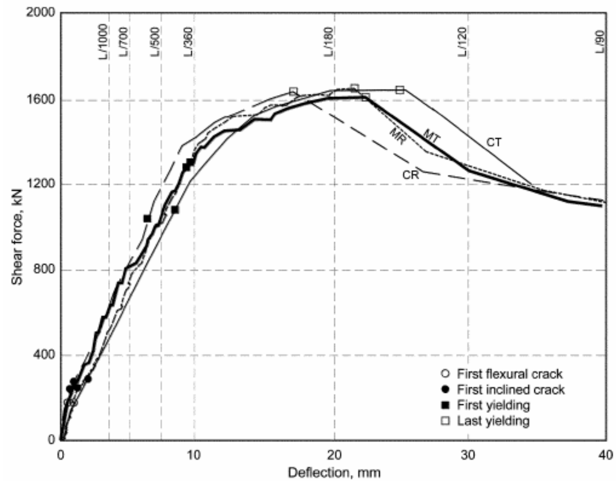
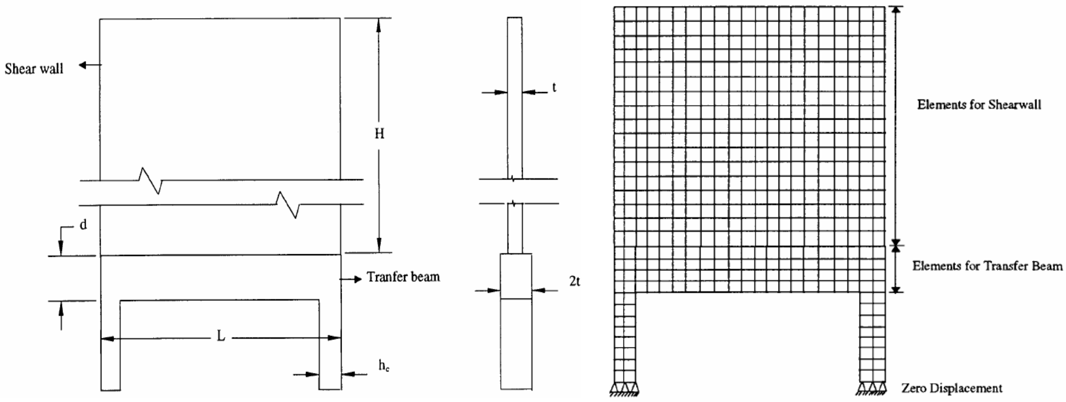
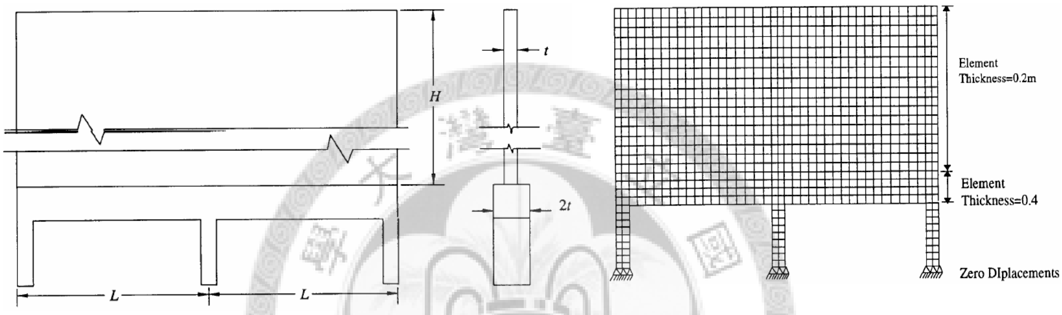


Figure 5.9-Shear force versus deflection curves for all beams during positive loading [79]



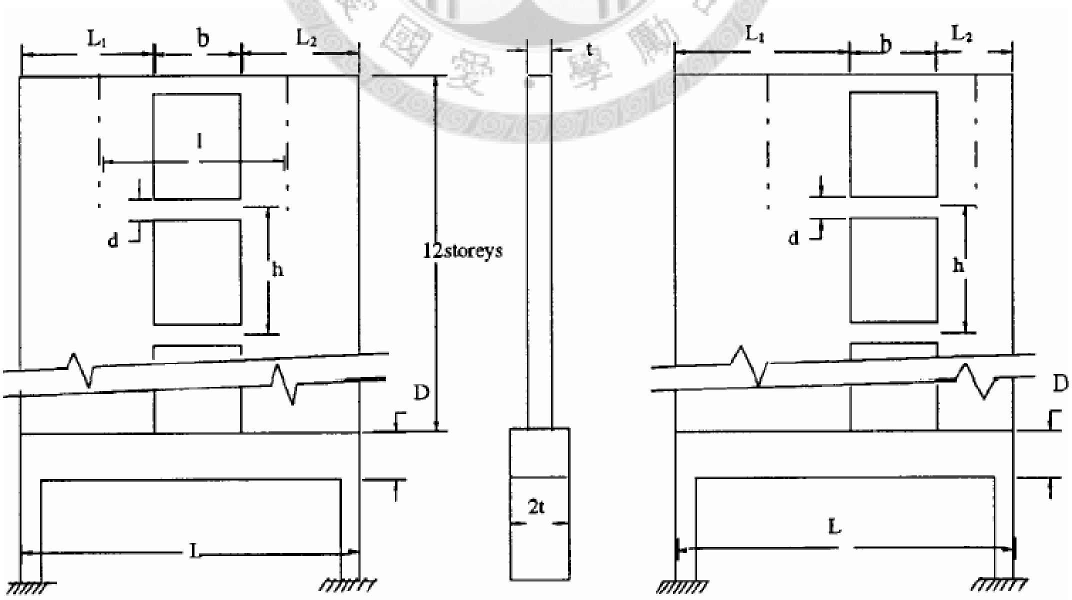
a) Typical transfer beam-shear wall system **b) Finite element model**

Figure 5.10-Single span transfer beams-shear walls system [80]



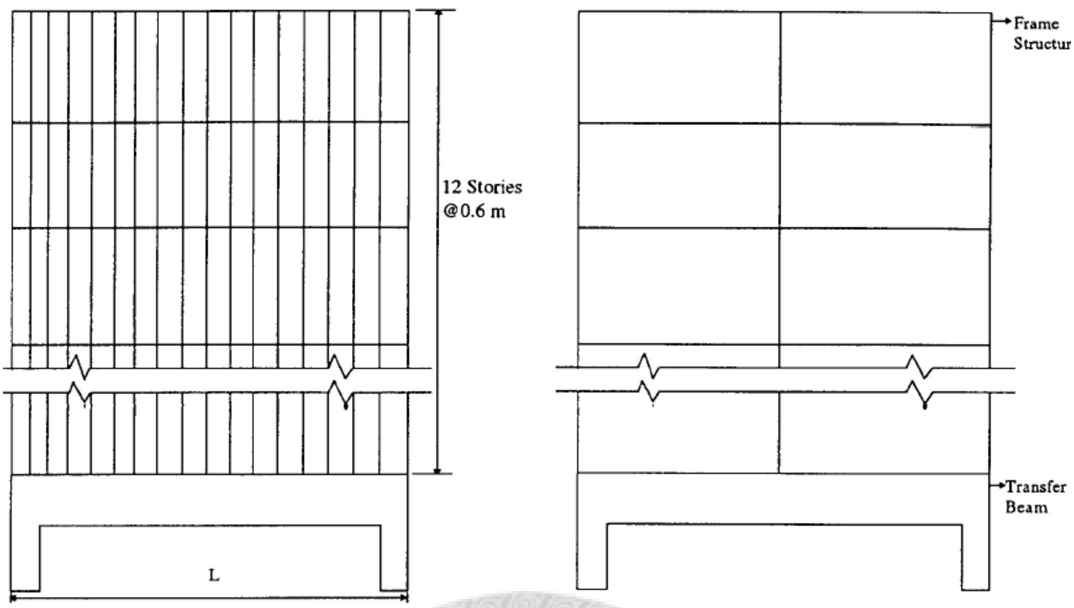
a) Typical two span transfer beam-shear wall system **b) Finite element model**

Figure 5.11-Two span transfer beams-shear walls system [80]



a) Coupled shear walls of equal width **b) Coupled shear walls of unequal width**

Figure 5.12-Typical transfer beams-coupled shear walls system [80]



a) Equivalent frame shear wall idealization

b) Simple two-bay frame

Figure 5.13-Typical transfer beams-frame system [80]

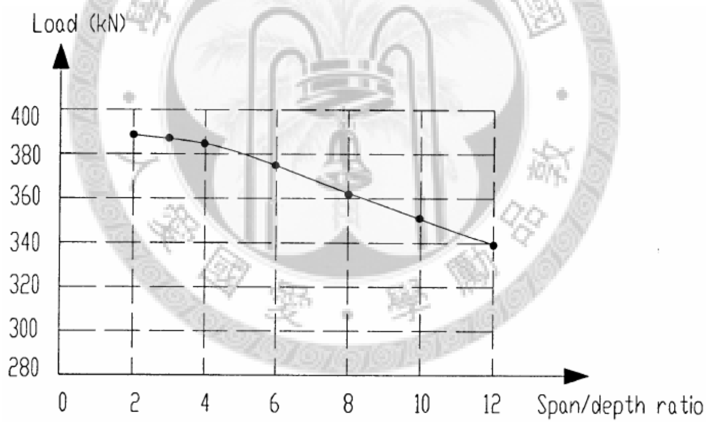


Figure 5.14-Variation of cracking load with different span-depth ratios [81]

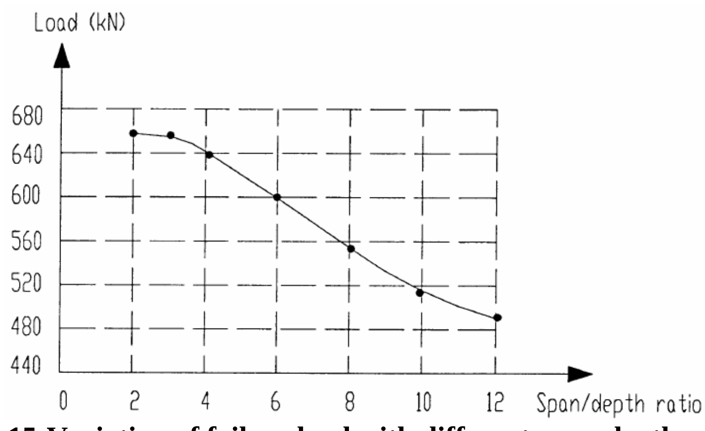


Figure 5.15-Variation of failure load with different span-depth ratios [81]

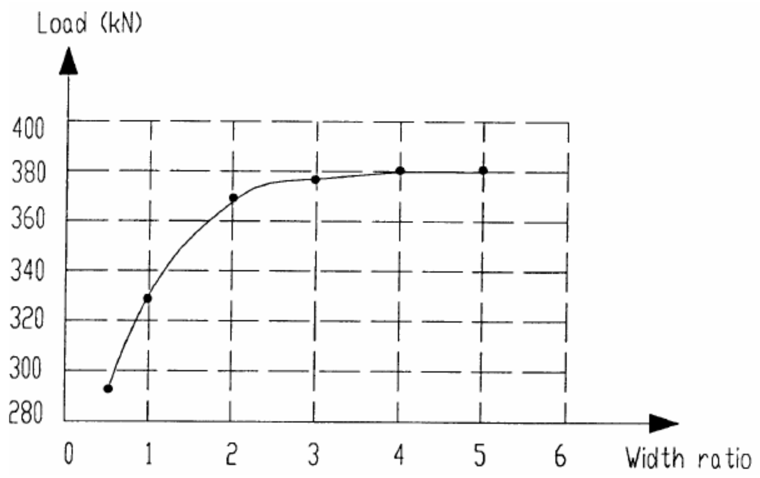


Figure 5.16-Variation of cracking load with different width ratios [81]

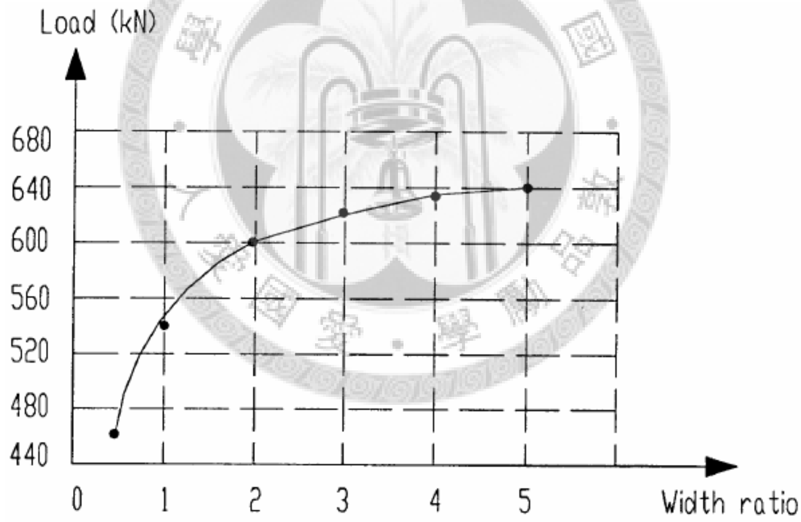
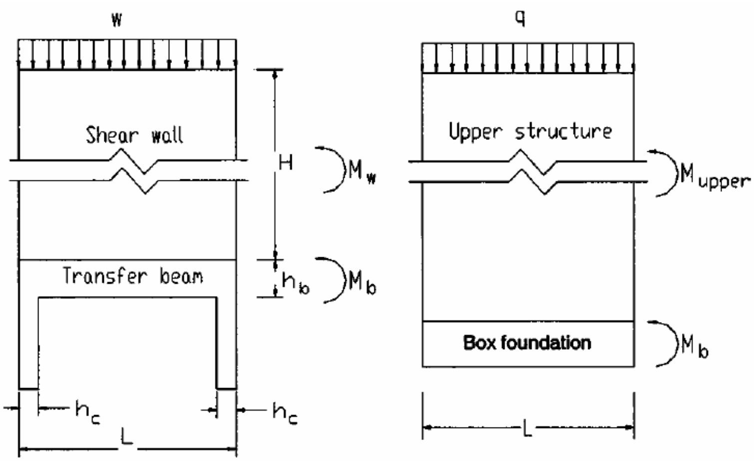
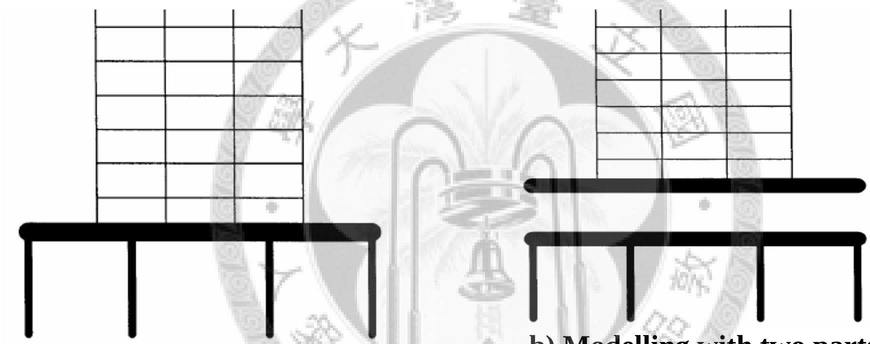


Figure 5.17-Variation of failure load with different width ratios [81]



a) Transfer beam-shear wall system b) Box foundation and upper structure

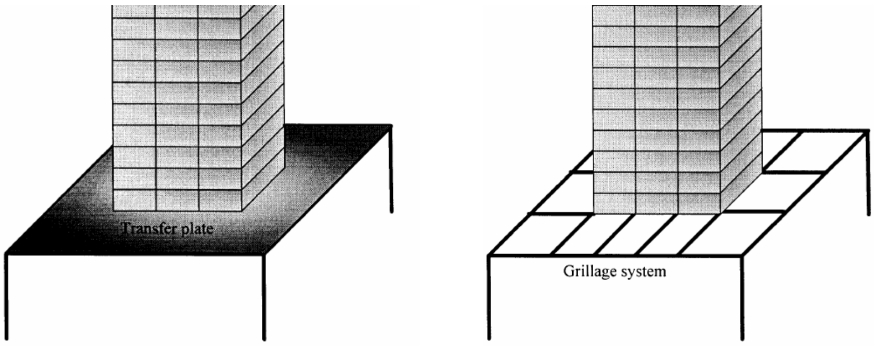
Figure 5.18-Box foundation analogy [82]



a) Transfer plate systems

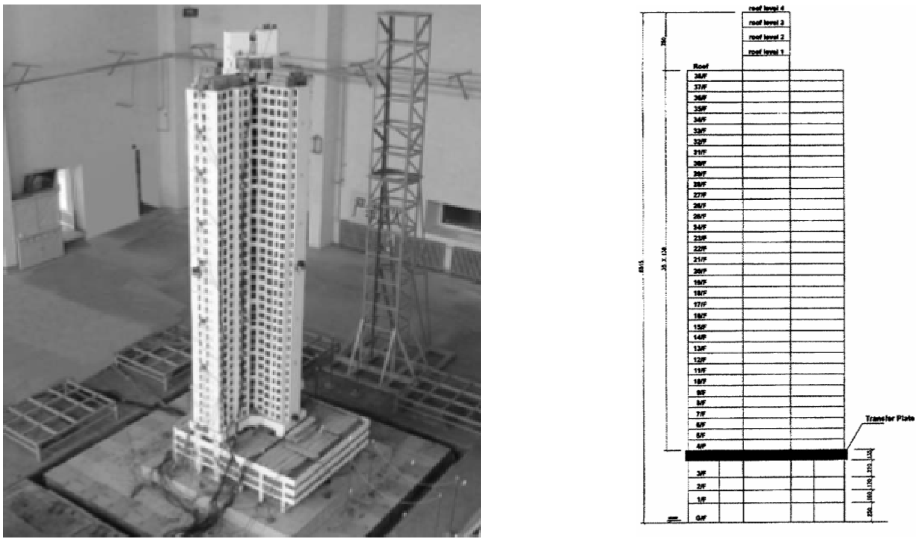
b) Modelling with two parts

Figure 5.19-Rigid base analogy method [84]



a) Transfer plate systems **b) Modelling with grillage system**

Figure 5.20-Grillage system method [84]



a) 1:20 scale model b) Elevation

Figure 5.21-Model of shaking table test [85]

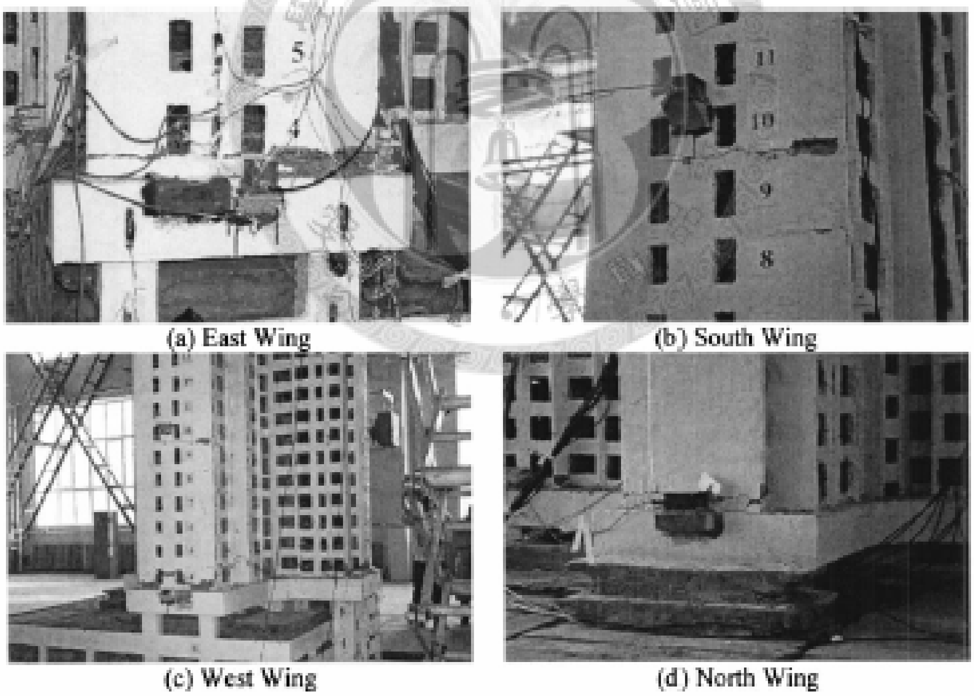
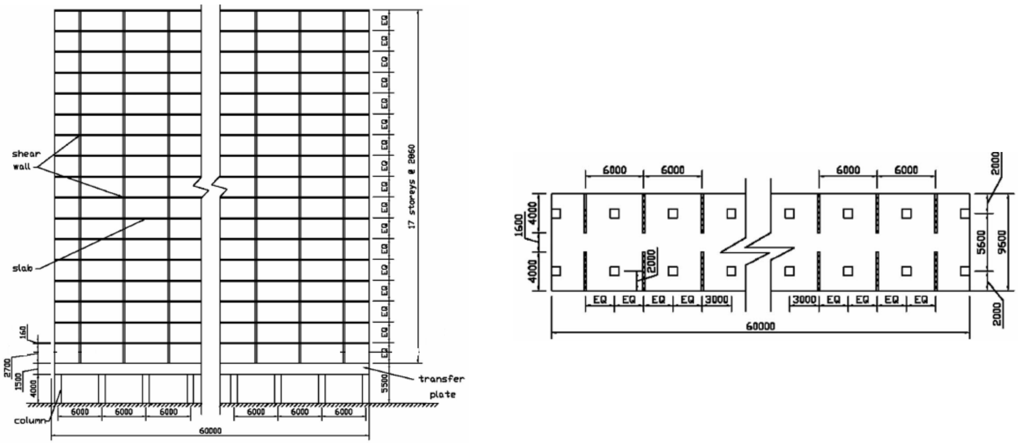


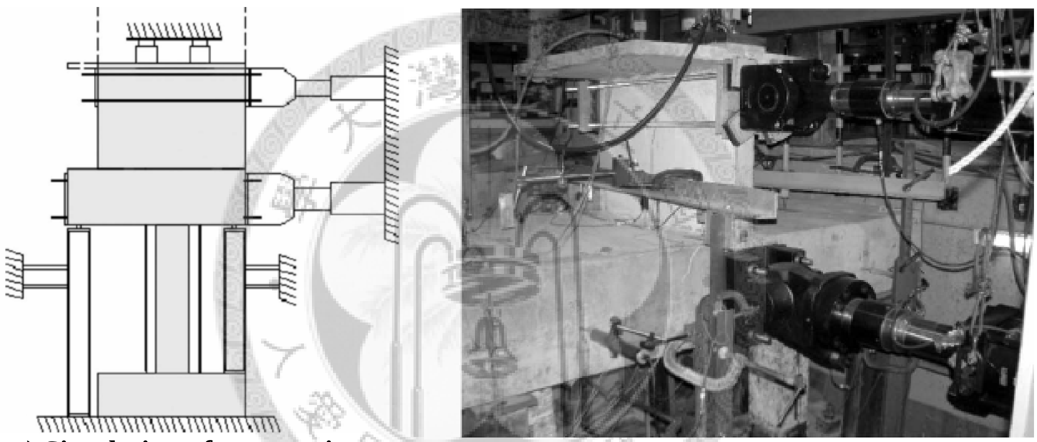
Figure 5.22-Mode of failure [85]



a) Elevated view

b) Plan view

Figure 5.23-Building with transfer plate [86]



a) Simulation of test specimen

b) Test specimen 1:20 scale

Figure 5.24-Experimental setup of test specimen [86]

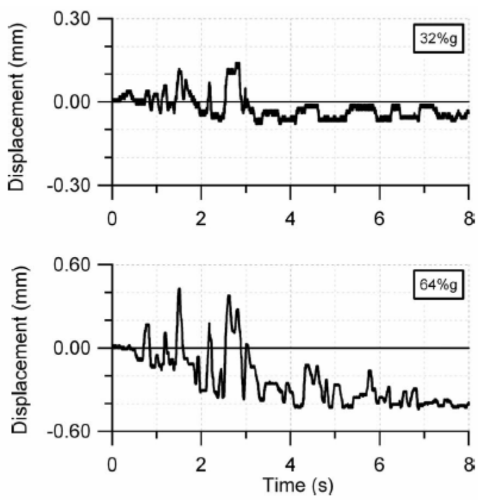


Figure 5.25-Vertical displacements at the transfer plate when subjected to El-Centro Earthquakes [86]

APPENDIX A

EXAMPLES ON CHECK FOR DEEP BEAMS USING STRUT-AND-TIE MODELS (ACI 318-08)

APPENDIX A
EXAMPLES ON CHECK FOR DEEP BEAMS
USING STRUT-AND-TIE MODELS (ACI 318-08)

Table A-1 through A-4 below show the calculation for one span deep beams of specimen data tested by Rogowsky et al. [62] (Figure A-1 through A-6)

Table A-1 Specimen No. 1/1.0N

Geometry parameters:

A	=	75	cm		C	=	45	cm
B	=	30	cm		D	=	100	cm
b	=	20	cm		a_o	=	5	cm
d	=	95	cm		l_b	=	20	cm
d'	=	0	cm		a	=	$A+l_b/2+B/2-B/4$	
w_t	=	10	cm		a	=	92.5	cm

Material data:

f'_c	=	266.22	kg/cm^2	
f_y	=	3600	kg/cm^2	(longitudinal bar)
f'_s	=	5700	kg/cm^2	(compr. steel)
f_{st}	=	5700	kg/cm^2	(stirrup)
E_s	=	2.04E+06	kg/cm^2	
E_c	=	244,743.74	kg/cm^2	$E_c = 15000(f'_c)^{(1/2)}$

Load test:

P_u	=	1,200.00	KN	120,000.00	kg
$V_u=P_u/2$	=	600	KN	60,000.00	kg

Reinforcement layout:

	Quan.	Dia.(mm)	$A_{-1bar}(cm^2)$	$A_s(cm^2)$		
Top bar	0	0	0.00	0.00	(A'_s)	$(r-1)A'_s$
Bottom bar	6	20	3.14	18.85	(A_s)	rA_s
Stirrup	4	6	0.28	1.13		
Horiz. Bar	0	0	0.00	0.00		

Result:

$n=E_s/E_c$	=	8.34					
$r=A_s/(bd)$	=	0.00992		$(y/2)*b*y+(r-1)A'_s(y-a_o) = rA_s(d-y)$			
$r'=A'_s/(bd)$	=	0.00000		y	=	12.41	cm
k	=	$\sqrt{(n\rho)^2+2n\rho}-n\rho$		k	=	y/d	
k	=	0.332	>	k	=	0.131	yes
jd=d-kd/3	=	84.477	cm	tg(q)	=	jd/a	
q	=	42.40	degree	sin(q)	=	0.674	
w_s	=	20.87	cm	cos(q)	=	0.738	

C-C-T node:

Struts: $F_{ns}=f_{ce}A_{cs} + A'_sf'_s$ (i) $f_{ce1}=0.85b_sf'_c$

f_{ce}	=	MIN(f_{ce1}, f_{ce2})		(ii) $f_{ce2}=0.85b_nf'_c$		
b_s	=	0.5		b_n	=	0.8 (CCT)
$b_{n\text{-}min}$	=	0.5				
A_{cs}	=	417.42	cm^2	A_{cs}	=	w_sb
f_{ce}	=	113.1435	kg/cm^2	f_{ce}	=	$0.85b_{n\text{-}min}f'_c$
F_{ns}	=	47,228.84	kg			
V_{ns}	=	31,849.17	kg	V_{ns}	=	$F_{ns}\sin(q)$

Ties: $F_{nt}=A_{ts}*f_y$

A_{ts}	=	18.85	cm^2	A_{ts}	=	A_s
F_{nt}	=	67,858.40	kg			
V_{nt}	=	61,972.70	kg	V_{nt}	=	$F_{nt}*\tan(q)$

Nodal Zones: $F_{nn}=f_{ce}A_{nz}$

b_n	=	0.8	(CCT)			
A_{nz}	=	417.42	cm^2	A_{nz}	=	w_sb
f_{ce}	=	181.0296	kg/cm^2	f_{ce}	=	$0.85*b_n*f'_c$
F_{nn}	=	75,566.14	kg			
V_{nn}	=	50,958.67	kg	V_{nn}	=	$F_{nn}*\sin(q)$

V_n = MIN{V_{ns},V_{nt},V_{nn}}

V_n = 31,849.17 kg

Check: $V_n <= V_u$

V_n = 31,849.17 kg

V_u = 60,000.00 kg

V_n = 31,849.17 < V_u = 60,000

Test/Code = 1.88 Conservative

1/1.0N

Table A-2 Specimen No. 1/1.0S

Geometry parameters:

A	=	75	cm	C	=	45	cm
B	=	30	cm	D	=	100	cm
b	=	20	cm	a_o	=	5	cm
d	=	95	cm	l_b	=	20	cm
d'	=	0	cm	a	=	$A+l_b/2+B/2-B/4$	
w_t	=	10	cm	a	=	92.5	cm

Material data:

f'_c	=	266.22	kg/cm^2	
f_y	=	3600	kg/cm^2	(longitudinal bar)
f'_s	=	5700	kg/cm^2	(compr. steel)
f_{st}	=	5700	kg/cm^2	(stirrup)
E_s	=	2.04E+06	kg/cm^2	
E_c	=	244,743.74	kg/cm^2	$E_c = 15000(f'_c)^{(1/2)}$

Load test:

P_u	=	1,400.00	KN	140,000.00	kg
$V_u=P_u/2$	=	700	KN	70,000.00	kg

Reinforcement layout:

	Quan.	Dia.(mm)	$A_{1bar}(cm^2)$	$A_s(cm^2)$		
Top bar	0	0	0.00	0.00	(A'_s)	$(r-1)A'_s$
Bottom bar	6	20	3.14	18.85	(A_s)	rA_s
Stirrup	4	6	0.28	1.13		
Horiz. Bar	0	0	0.00	0.00		

Result:

$n=E_s/E_c$	=	8.34					
$r=A_s/(bd)$	=	0.00992		$(y/2)*b*y+(r-1)A'_s(y-a_o) = rA_s(d-y)$			
$r'=A'_s/(bd)$	=	0.00000		y	=	12.41	cm
k	=	$\sqrt{(n\rho)^2+2n\rho}-n\rho$		k	=	y/d	
k	=	0.332	>	k	=	0.131	yes
jd=d-kd/3	=	84.477	cm	tg(q)	=	jd/a	
q	=	42.40	degree	sin(q)	=	0.674	
w_s	=	20.87	cm	cos(q)	=	0.738	

C-C-T node:

Struts: $F_{ns}=f_{ce}A_{cs} + A'_sf'_s$ (i) $f_{ce1}=0.85b_sf'_c$

f_{ce} = $MIN(f_{ce1}, f_{ce2})$ (ii) $f_{ce2}=0.85b_nf'_c$

b_s = 0.4 b_n = 0.8 (CCT)

$b_{n\text{-}min}$ = 0.4

A_{cs} = 417.42 cm^2 $\quad$ $A_{cs} = w_s b$

f_{ce} = 90.5148 kg/cm^2 $\quad$ $f_{ce} = 0.85 b_{n\text{-}min} f'_c$

F_{ns} = 37,783.07 kg

V_{ns} = 25,479.34 kg $\quad$ $V_{ns} = F_{ns}\sin(q)$

Ties: $F_{nt}=A_{ts}*f_y$

A_{ts} = 18.85 cm^2 $\quad$ $A_{ts} = A_s$

F_{nt} = 67,858.40 kg

V_{nt} = 61,972.70 kg $\quad$ $V_{nt} = F_{nt}*\tan(q)$

Nodal Zones: $F_{nn}=f_{ce}A_{nz}$

b_n = 0.8 (CCT)

A_{nz} = 417.42 cm^2 $\quad$ $A_{nz} = w_s b$

f_{ce} = 181.0296 kg/cm^2 $\quad$ $f_{ce} = 0.85*b_n*f'_c$

F_{nn} = 75,566.14 kg

V_{nn} = 50,958.67 kg $\quad$ $V_{nn} = F_{nn}*\sin(q)$

V_n = $\mathrm{MIN}\{V_{ns}, V_{nt}, V_{nn}\}$

V_n = 25,479.34 kg

Check: $V_n \le V_u$

V_n = 25,479.34 kg

V_u = 70,000.00 kg

V_n = 25,479.34 < V_u = 70,000 Sastified

Test/Code = $\boxed{2.75}$ Too conservative

1/1.0S

Table A-3 Specimen No. 2/1.0N

Geometry parameters:

A	=	75 cm		C	=	45 cm
B	=	30 cm		D	=	100 cm
b	=	20 cm		a_o	=	5 cm
d	=	95 cm		l_b	=	20 cm
d'	=	98 cm		a	=	$A+l_b/2+B/2-B/4$
w_t	=	10 cm		a	=	92.5 cm

Material data:

f'_c	=	273.36 kg/cm^2				
f_y	=	3600 kg/cm^2	(longitudinal bar)			
f'_s	=	5700 kg/cm^2	(compr. steel)			
f_{st}	=	5700 kg/cm^2	(stirrup)			
E_s	=	2.04E+06 kg/cm^2				
E_c	=	248,004.03 kg/cm^2		E_c	=	$15000(f'_c)^{(1/2)}$

Load test:

P_u	=	1,500.00 KN	150,000.00 kg
$V_u=P_u/2$	=	750 KN	75,000.00 kg

Reinforcement layout:

	Quan.	Dia.(mm)	$A_{\text{-}1bar}$(cm^2)	A_s(cm^2)		
Top bar	2	6	0.28	0.57	(A'_s)	$(r-1)A'_s$
Bottom bar	6	20	3.14	18.85	(A_s)	rA_s
Stirrup	4	6	0.28	1.13		
Horiz. Bar	4	6	0.28	1.13		

Result:

$n=E_s/E_c$	=	8.23				
$r=A_s/(bd)$	=	0.00992		$(y/2)*b*y+(r-1)A'_s(y-a_o) = rA_s(d-y)$		
$r'=A'_s/(bd)$	=	0.00030		y	=	12.45 cm
k	=	$\sqrt{(n\rho)^2+2n\rho}-n\rho$		k	=	y/d
k	=	0.331	>	k	=	0.131 yes
$jd=d-kd/3$	=	84.533 cm		tg(q)	=	jd/a
q	=	42.42 degree		sin(q)	=	0.675
w_s	=	20.87 cm		cos(q)	=	0.738

C-C-T node:

Struts:		$F_{ns}=f_{ce}A_{cs} + A'_sf'_s$	(i) $f_{ce1}=0.85b_sf'_c$		
f_{ce}	=	$MIN(f_{ce1}, f_{ce2})$	(ii) $f_{ce2}=0.85b_nf'_c$		
b_s	=	0.6	b_n	=	0.8 (CCT)

b_{n-min} = 0.6

A_{cs} = 417.48 cm^2 A_{cs} = $w_s b$

f_{ce} = 139.4136 kg/cm^2 f_{ce} = $0.85 b_{n-min} f'_c$

F_{ns} = 61,425.22 kg

V_{ns} = 41,437.49 kg V_{ns} = $F_{ns} sin(q)$

Ties: $F_{nt}=A_{ts}*f_y$

A_{ts} = 18.85 cm^2 A_{ts} = A_s

F_{nt} = 67,858.40 kg

V_{nt} = 62,013.52 kg V_{nt} = $F_{nt}*tan(q)$

Nodal Zones: $F_{nn}=f_{ce}A_{nz}$

b_n = 0.8 (CCT)

A_{nz} = 417.48 cm^2 A_{nz} = $w_s b$

f_{ce} = 185.8848 kg/cm^2 f_{ce} = $0.85*b_n*f'_c$

F_{nn} = 77,602.60 kg

V_{nn} = 52,350.76 kg V_{nn} = $F_{nn}*sin(q)$

V_n = $MIN\{V_{ns},V_{nt},V_{nn}\}$

V_n = 41,437.49 kg

Check: $V_n <= V_u$

V_n = 41,437.49 kg

V_u = 75,000.00 kg

V_n = 41,437.49 $<$ V_u = 75,000 Sastified

Test/Code = 1.81 Conservative

2/1.0N

Table A-4 Specimen No. 2/1.0S

Geometry parameters:

A	=	75	cm	C	=	45	cm
B	=	30	cm	D	=	100	cm
b	=	20	cm	a_o	=	5	cm
d	=	95	cm	l_b	=	20	cm
d'	=	98	cm	a	=	$A+l_b/2+B/2-B/4$	
w_t	=	10	cm	a	=	92.5	cm

Material data:

f'_c	=	273.36	kg/cm^2	
f_y	=	3600	kg/cm^2	(longitudinal bar)
f'_s	=	5700	kg/cm^2	(compr. steel)
f_{st}	=	5700	kg/cm^2	(stirrup)
E_s	=	2.04E+06	kg/cm^2	
E_c	=	248,004.03	kg/cm^2	$E_c = 15000(f'_c)^{(1/2)}$

Load test:

P_u	=	1,500.00	KN	150,000.00	kg
$V_u=P_u/2$	=	750	KN	75,000.00	kg

Reinforcement layout:

	Quan.	Dia.(mm)	$A_{-1bar}(cm^2)$	$A_s(cm^2)$		
Top bar	2	6	0.28	0.57	(A'_s)	$(r-1)A'_s$
Bottom bar	6	20	3.14	18.85	(A_s)	rA_s
Stirrup	0	0	0.00	0.00		
Horiz. Bar	4	6	0.28	1.13		

Result:

$n=E_s/E_c$	=	8.23					
$r=A_s/(bd)$	=	0.00992		$(y/2)*b*y+(r-1)A'_s(y-a_o) = rA_s(d-y)$			
$r'=A'_s/(bd)$	=	0.00030		y	=	12.45	cm
k	=	$\sqrt{(n\rho)^2+2n\rho}-n\rho$		k	=	y/d	
k	=	0.331	>	k	=	0.131	yes
$jd=d-kd/3$	=	84.533	cm	tg(q)	=	jd/a	
q	=	42.42	degree	sin(q)	=	0.675	
w_s	=	20.87	cm	cos(q)	=	0.738	

C-C-T node:

Struts: $F_{ns}=f_{ce}A_{cs} + A'_sf'_s$ (i) $f_{ce1}=0.85b_sf'_c$

f_{ce} = $MIN(f_{ce1}, f_{ce2})$ (ii) $f_{ce2}=0.85b_nf'_c$

b_s = 0.5 b_n = 0.8 (CCT)

b_{n-min}	=	0.5				
A_{cs}	=	417.48	cm^2	A_{cs}	=	$w_s b$
f_{ce}	=	116.178	kg/cm^2	f_{ce}	=	$0.85 b_{n-min} f'_c$
F_{ns}	=	51,724.90	kg			
V_{ns}	=	34,893.65	kg	V_{ns}	=	$F_{ns} \sin(q)$

Ties: $F_{nt}=A_{ts}*f_y$

A_{ts}	=	18.85	cm^2	A_{ts}	=	A_s
F_{nt}	=	67,858.40	kg			
V_{nt}	=	62,013.52	kg	V_{nt}	=	$F_{nt}*\tan(q)$

Nodal Zones: $F_{nn}=f_{ce}A_{nz}$

b_n	=	0.8	(CCT)			
A_{nz}	=	417.48	cm^2	A_{nz}	=	$w_s b$
f_{ce}	=	185.8848	kg/cm^2	f_{ce}	=	$0.85*b_n*f'_c$
F_{nn}	=	77,602.60	kg			
V_{nn}	=	52,350.76	kg	V_{nn}	=	$F_{nn}*\sin(q)$

V_n = $\mathrm{MIN}\{V_{ns}, V_{nt}, V_{nn}\}$

V_n = 34,893.65 kg

Check: $V_n <= V_u$

V_n	=	34,893.65	kg				
V_u	=	75,000.00	kg				
V_n	=	34,893.65	<	V_u	=	75,000	Sastified
Test/Code	=	$\boxed{2.15}$	Too conservative				

2/1.0S

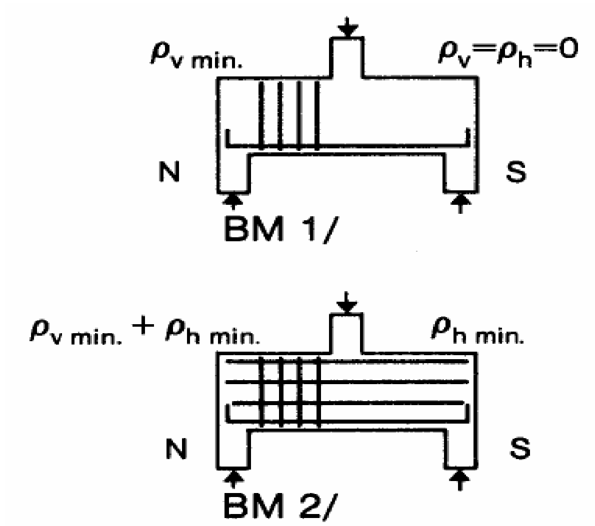


Figure A-1. Typical test series (Beam BM1 and BM2)

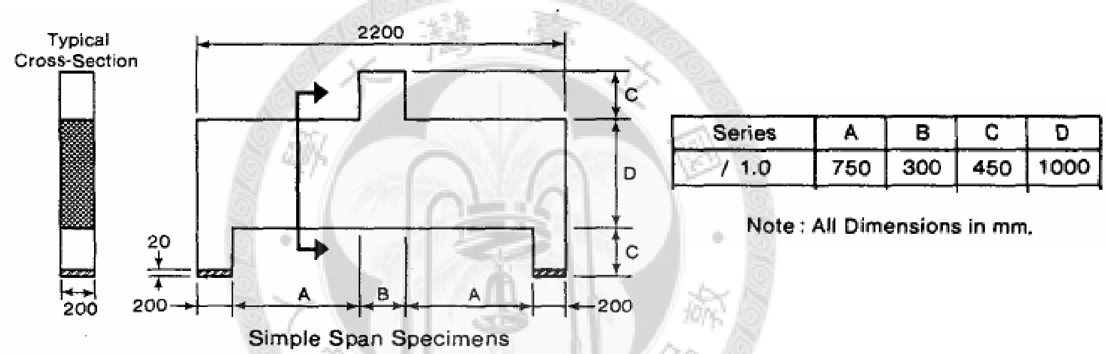


Figure A-2. Overall dimensions of specimens

		Top steel			Bottom steel			Web steel*	
Specimen	f'_c, MPa	Bars	$A_s f_y$ per bar, kN	d, mm	Bars	$A_s f_y$ per bar, kN	d, mm	Number of stirrups	Number of horizontal bars
1/1.0N	26.1	—	—	—	6-20M	114	950	4	—
1/1.0S	26.1	—	—	—	6-20M	114	950	—	—
2/1.0N	26.8	2-6 mm	16.2	980	6-20M	114	950	4	4
2/1.0S	26.8	2-6 mm	16.2	980	6-20M	114	950	—	4

Figure A-3. Details of specimens

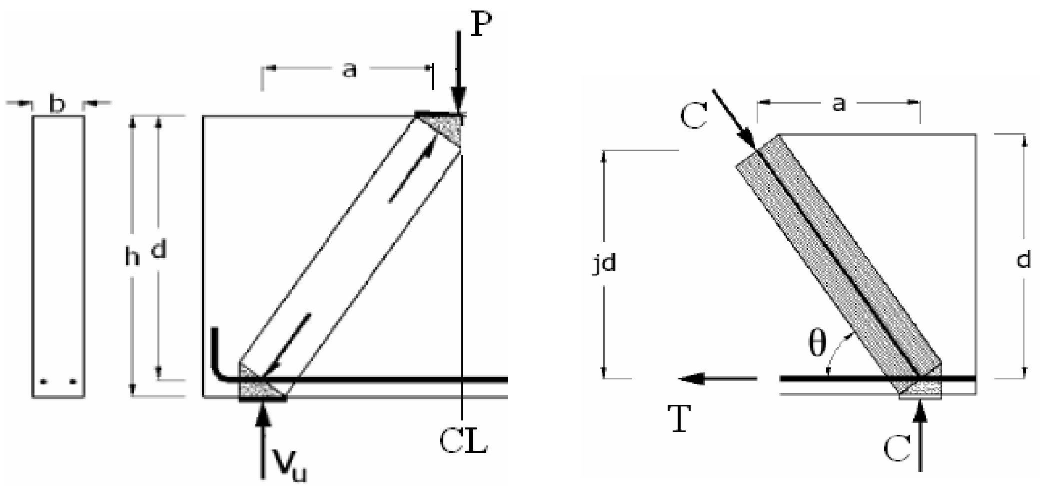
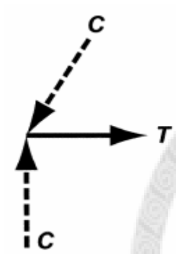
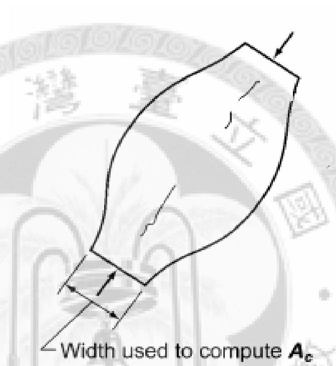


Figure A-4. Geometry parameters of deep beams



a) C-C-T Node



b) Bottle-shape strut

Figure A-5. Nodal types for checking

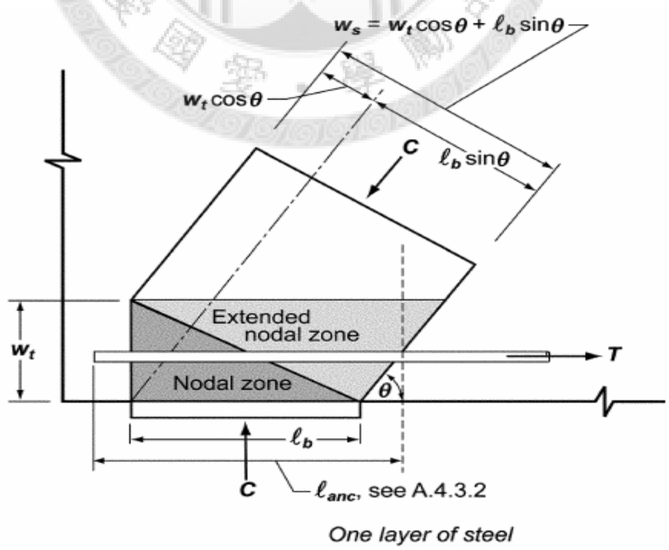


Figure A-6. Nodal zone for calculation of A_{cs}, A_{nz}