PERFORMANCE OF REINFORCED CONCRETE BEAM-COLUMN JOINT SUBJECTED TO REPEATED REVERSED LOAD

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SHORT TERM GRANT

NO. VOT : 0777

UNIVERSITI TUN HUSSEIN ONN MALAYSIA

ABSTRACT

Beam-column joints of reinforced concrete structures are common exterior moment resisting frame in buildings. Understanding the complex behavior of this joint method under repeated load is crucial because of their basic materials have limited strengths that can cause the limited force carrying capacity. This study was focus on the performance of beam column joint subjected to repeated reversed load. Three specimens of reinforced concrete (RC) exterior beam-column joint with different steel detail arrangement had been prepared and tested under repeated reversed load. All beam-column joint specimens were designed in accordance to BS 8110 and the types of joint were exterior joint. First specimen was the control specimen where no additional shear link on beam reinforcement and no anchorages of bars were tied between beam and column. Second speciment consist of additional shear link on beam reinforcement with 75 mm spacing which is closer then specimen 1 and 3. Third specimen introduced cross-bracing of reinforcement bars at the intersection of beam-column joint. In this study, failure load, beam displacement, stiffness, strain value of steel reinforcement, mode of failure had been presented. The repeated reversed loads were applied to the end of cantilever beam. The dimension for each column was 300 x 300 x 1500 mm, and the cantilever beam with dimension, 300 x 150 x 1000 mm was located at the mid-height of column. High-yield steel was utilized for all main reinforcement with 8T16 bars for column rebar and 4T16 for cantilever RC beam. Mild steel type was used for all steel links. In addition, 30 grade of concrete was employed and the concrete design was based on JKR mix design method. The result of this study shown that the best performances of RC beam column joint is third specimen which is can be explained that cross bracing joint of the beam-column joint is the greatest type of joint among the three specimen of joint since it showed the well performance under seismic condition.

ABSTRAK

Sambungan rasuk-tiang struktur konkrit bertetulang adalah penanggung momen kerangka dalam bangunan. Memahami sifat kompleks sambungan rasuk tiang di bawah beban berulang kali adalah penting kerana bahan-bahan asas yang digunakan mempunyai kekuatan yang terhad yang boleh menyebabkan keupayaan terhad menanggung beban. Kajian ini adalah focus kepada mengkaji prestasi sambungan rasuk tiang di bawah beban ulang alik. Tiga spesimen konkrit bertetulang (RC) sambungan rasuk tiang luaran dengan susunan keluli yang berbeza telah disediakan dan diuji di bawah beban ulang alik. Semua spesimen sambungan rasuk-tiang telah direkabentuk mengikut BS 8110 dengan jenis sambungan luaran. Spesimen pertama merupakan spesimen kawalan yang mana tiada sebarang tetulang tambahan antara rasuk dan tiang. Spesimen kedua terdiri daripada tambahan perangkai ricih pada rasuk dengan 75 mm jarak yang lebih rapat daripada sampel 1 dan 3. Spesimen ketiga diperkenalkan tetulang bersilang pada sambungan rasuk-tiang. Dalam kajian ini, beban gagal, anjakan rasuk, nilai terikan tetulang keluli, mod kegagalan telah dibentangkan. Beban ulang alik telah digunakan pada hujung rasuk julur. Dimensi bagi setiap lajur adalah 300 x 300 x 1500 mm, dan rasuk julur dengan dimensi 300 x 150 x 1000 mm yang terletak di tengah tiang. Keluli kekuatan tinggi telah digunakan untuk semua tetulang utama dengan bar 8T16 untuk tetulang tiang dan 4T16 untuk rasuk julur. Jenis keluli lembut telah digunakan sebagai perangkai. Di samping itu, konkrit gred 30 direkabentuk berdasarkan kepada kaedah rekabentuk campuran JKR. Hasil kajian ini menunjukkan bahawa prestasi terbaik sambungan rasuk tiang adalah spesimen ketiga. Di sini dapat dijelaskan bahawa tetulang bersilang pada sambungan rasuk-tiang adalah jenis terbaik dalam menanggung beban seismik.

CONTENTS

CHAPTER 1

INTRODUCTION

1.1   General

The joints between beams and columns are critical components in reinforced concrete structures. According to Haach (2007) beam-column joints are critical regions of structures due to the fact that they are located in an area, where the bond and normal stresses are substantially high. The performance of beam-column joint is influenced by many parameters such as material used, column load, and detailing arrangement of the column and beam steel. The RC structure is separated to individual structure component including column-beam element, wall element, floor and connection between these elements. Compressive strength of concrete, detailing of beam-column joints and workmanship play an important role in assessing the seismic performance under seismic loading (Paulay *et al.*, 1978). Beam-column joint is defined as the zone of intersection between beams and columns with the functional requirement which enable the adjoining members to develop and sustain their ultimate capacity. The joint should provide sufficient strength and endurance to resist the internal forces transferred by the framing members. All structures are subjected to repeated loads, called cyclic/reversed load, the failure area that always occurred on RC structure is at the connection between beam and columns. Due to complexity in repairing of the buildings damaged in beam-column joints in case of the seismic attack and structural, the research for beam-column joint should be done from time to time although the countries are not located in seismic zone. In this study the connection specimen of concrete beam-column was design and built up with the reinforced concrete beam projecting from column and was tested under repeated

reverse load. Basically, there are three types of beam-column joints which are interior joint, exterior joint and corner joint (Uma and Meher, 2002). This study focuses on the design, construction and testing exterior beam-column joint. Consequently, three identical half-scale beam-column joints with three types of joint arrangements were constructed, calibrated and tested in heavy structural laboratory. All three specimens were design accordance to BS 8110. The first sample was no additional bar or link as control specimen. Second specimen was introduced additional shear link at beam reinforcement. Third specimen was located additional cross-bracing of reinforcement bars at the intersection of beam-column joint. Results of three specimens were compared to see the best performing of beam column joint.

1.2   Problem Statement

Usually beam-column joint have a problem when forces larger than design load are applied. Cyclic load or repeated reversed load can be able to mark by a combination of large shear forces, diagonal tension and high bond stresses in the reinforcement bars. Since the 1960s, many experimental and theoretical studies have been conducted to investigate and form the seismic resistance of beam-column joint. The deficiencies of joints are mainly caused by inadequate transverse reinforcement and insufficient anchorage capacity in the joint. These problems have been highlighted by the damage observed in recent devastating earthquakes in different countries. Evidence from recent earthquakes, such as the 1995 Hyogo-ken Nanbu (Japan), the 1999 Kocaeli (Turkey) and Chi-Chi (Taiwan) earthquakes, shows the total collapse of many structures caused by brittle shear failure in the joint. Most of these joint brittle shear failures were due to non-ductile performance, either poor anchorage of the main reinforcing bars or simply inadequate transverse reinforcement in the joints, of reinforced concrete moment-resisting frames. In the 1995 Hyogo-ken Nanbu earthquake, a phenomenon was found that reinforced concrete buildings built in the pre-1970s suffered more severely than those built after the development of current seismic codes. A damaged structure after the Kocaeli earthquake is shown in Figure 1.1 demonstrating a good example of this failure mode.

Figure 1.1: Damaged beam-column joint during earthquake (1999 Kocaeli, Turkey)

Repairing damaged joints is difficult, and so damage must be avoided. Thus, beam-column joints must be designed properly to resist rapidly massive load which over then design load and tested to see the performance of joint. Because of rapidly massive load which over then design load, the beams bordering a joint are subjected to rotation of moments in the same (clockwise or counterclockwise) direction. Due to these moments, the top bars in the beam-column joint are pulled in one direction and the bottom ones in the opposite direction. These forces are balanced by bond stress developed between concrete and steel in the joint region.

Generally the concrete cracking and joint destruction can be controlled by several factors which provide a larger column size, provide additional anchorage bar at joint, provide a bond that binds tightly and closed the bar for the continuation of the pole in the area. This bond will jointly hold the concrete in the joint and also resist shear forces, thereby reducing cracking and disintegration of the concrete.

In addition, normally in seismic regions the design of beam-column joints is an important part of earthquake resistant design for reinforced concrete moment-resisting frames. However in non-seismic regions especially Malaysia, structures are mainly designed to resist gravity loads with little consideration of the effect of lateral loads. Although Malaysia are not located in seismic zone, these structures can be subjected to lateral loads from the long distance earthquake or explosion, for example

the location and environment of Malaysia are close to Indonesia which is one of seismic regions. Therefore the further research must be done to improve the performance especially the anchorage capacity in the joint. Most recently the 2004 Sumatra Earthquake in the Andaman Sea, recorded at 9.3 moment magnitude, caused violent shaking of many buildings in Bangkok, though the epicenter was more than 800 kilometers away. The quake has prompted a serious public concern on seismic safety of buildings. In the Southeast Asian countries, there are many low-rise and mid-rise buildings of up to 10 stories constructed as beam-column rigid frames without earthquakes resistant design. The framestructures mainly resist lateral forces through bending ofbeams and columns. Most of these structures were designed for gravity load only according to the American Concrete Institute.s (ACI) building code in Thailand and British Standard (BC) code in Singapore and Malaysia.

1.3   Objective of Study

There are Three objectives needed to be achieved in this study:

1. To study the performance of reinforced concrete beam-column joint under repeated reversed load.
2. To identify the microscopic physical damage of RC beam-column joints under repeated reverse load.
3. To investigate the maximum load, beam displacement, stiffness and strain under repeated reverse load.

1.4   Scope of study

The type of beam-column joint in this study was exterior joint. The scope that were cover as below:

a. To evaluate and compare the performance of three beam column joint specimens with different bar arrangement.

b. The parameter had been determined were displacement, strain value of concrete and steel, maximum load, and mode of failure at beam column joint.

c. The strain gauge was positioned on main bar, link and concrete to monitor the strain reading.

d. The LVDT was positioned at four point of cantilever beam to monitor the displacements of beam.

e. All three specimens were design accordance to BS 8110 which is non-seismically design detailed.The first sample was no additional bar or link as control specimen. Second specimen was introduced additional shear link at beam reinforcement. Third specimen was located additional cross-bracing of reinforcement bars at the intersection of beam-column joint. Results of three specimens were compared to see the best performing of beam column joint.

f. Dimension for each column were 300 x 300 x 1500 mm, at mid-height of column was located the cantilever beam with dimension 1000 mm long , 300 mm depth and 150 mm wide. All main reinforcement was high-yield steel, with eight T16 bars was used for the column reinforcement and a four of T16 bars for the cantilever reinforced concrete beam at centre of column. Links was all mild steel.Grade 30 of concrete was utilized and the concrete design was based on JKR mix design method.

g. Testing for repeated reversed load test was conducted when the specimens achieve the age of 28 days. The method of testing was according to previous research which is done by Rajaram, Murugesan, and Thirugnanam, (2010)

1.5   Significant of Study

This research is carried out to understand the complex mechanisms and safe behavior of existing beam column joints design at non-seismic regions especially in Malaysia as a carefulness step in case of earthquakes occur, this is because the location and environment of Malaysia are closely to Indonesia which is one of seismic regions and this research are useful for the next related research. The additional bar on beam column joint should be increase the performance of beam column joint. The effects of this cure technique in increasing load carrying capacity and in improving the behavior of RCbeam-column joints. In additional, the influence of anchorages bar to the existing RC beam-column joint was also studied.

CHAPTER 2

LITERATURE REVIEW

2.1 Introduction

The seismic exposure of old existing reinforced concrete buildings constructed in zones of low to medium seismicity was recently discussed by several researchers in the USA and New Zealand (Aycardi et al., 1994, Bracci et al., 1995, El-attar et al., 1997, Hakuto et al., 2000). Even in the South East Asian region such as Thailand, Singapore and Malaysia, which was usually believed to be safe against seismic hazard, the research in this issue has gained more attention (Li et al., 2002, Li and Pan, 2004, Warnichai, 2004). A beam-column joint is the critical zone in reinforced concrete buildings where vertical loading and lateral loading are met and transfer their load to the foundation. This type of joint has high risk of failures as compared to others structural components when an earthquake strikes at any areas in seismic regions where this is the possibility of occurrence of plastic-hinge zone mechanism. Beam column joint in Reinforce Concrete buildings is a segment of columns that are common to beams at their intersections (Nilson et al., 2004). The most important joint are of course beam-to-column and beam-to-beam Reinforced concrete (RC) connections. The function of a beam-column joint in a frame is to transfer the loads and moments at the ends of the beams into the columns. Joints are often the weakest links in a structural system. The joints should have adequate strength and stiffness to resist the internal forces induced by the framing members. Much valuable work has been done in this area very recently. However, our understanding of joint behavior and of existing detailing practice is still in need of much improvement. Joint behavior is especially critical for structures subject to

earthquake effects. The shear forces developed as a result of such an excitation should be safely transferred through joints. According to Viest et al. (1997), connections or joints are potentially the most critical and possibly the least understood parts of the structural frame. Today, beam-columns joint becomes a crucial awareness of engineers because of vibration or movement to the structure from effects of earthquakes and explosions that result in structural failure and collapse. Therefore, various methods and measures have been made to ensure that the connection on the structure which will not fail and cause harm to the users. The materials of Reinforce concrete (RC) as well have limited strengths so the joints have limited force carrying capacity. When load are larger than allowable capacity due to huge load or cyclic load, joints are severely damaged. Repairing damaged joints is complicated, and so damage must be evaded. Thus, beam-column joints must be designed to defend against earthquake effects. (Murty, 2005)

2.2   Types of beam column connection

The joint is defined as the portion of the column within the depth of the deepest beam that frames into the column. In a moment resisting frame, three types of joints can be identified such as interior joint, exterior joint and corner joint. Interior joint condition is when four beams frame into the vertical faces of a column, the joint is called as an interior joint. Meanwhile Exterior joint condition is when one beam frames into a vertical face of the column and two other beams frame from perpendicular directions into the joint, then the joint is called as an exterior joint. Corner joint condition is when a beam each frames into two adjacent vertical faces of a column, then the joint is called as a corner joint .(Rajaram, Murugesan and Thirugnanam, 2010)

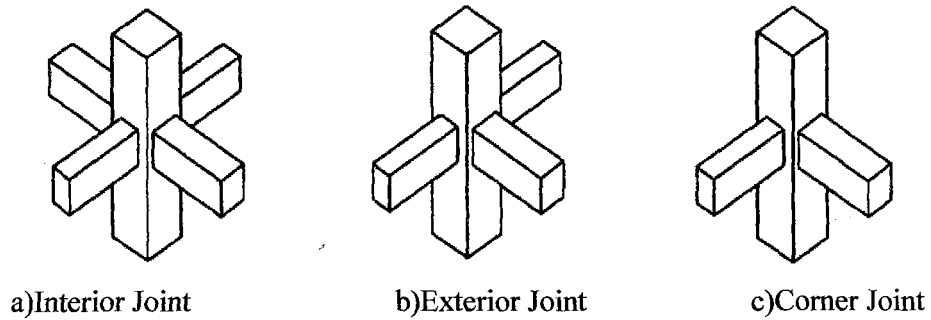a)Interior Joint   b)Exterior Joint   c)Corner Joint

Figure 2.1: Types of joints (Uma and Meher, 2002)

Previous research by Salim (2007) indicated that the severity of forces and demands on the performance of these joints calls for greater understanding of their seismic behavior. These forces develop complex mechanisms involving bond and shear within the joint. In building structures, the joint also can be categorized as weak, moderate, intermediate and strong joint.

i. Weak joint - Joints designed prior to the 1970's. Typically, these joints have minimal amounts, if any, of transverse reinforcement in the joint.

ii. Moderate joint – Joints designed between 1970 and 1990. These joints have a nominal amount of transverse reinforcement, enough to sustain concrete cracking without significant strength loss.

iii. Intermediate joint – Joints that have a nominal amount of transverse reinforcement, enough to sustain concrete cracking, but not enough to sustain yielding of the framing members. Bar yielding may be precluded by the lack of standard hooks, or by insufficient anchorage length for column bars passing through the joint.

iv. Strong joint - Joints designed after 1990, containing significant amounts of horizontal and vertical reinforcement in the joint to enable proper confinement of the joint core and provide the necessary mechanisms for force transfer.

2.3   Loading

Loading on tall buildings is different from low-rise buildings in many ways such as large accumulation of gravity loads on the floors from top to bottom, increased significance of wind loading and greater importance of dynamic effects. Thus, multi-storey structures need correct assessment of loads for safe and economical design. Excepting dead loads, the assessment of loads cannot be done accurately. Live loads can be anticipated approximately from a combination of experience and the previous field observations. But, wind and earthquake loads are random in nature. It is difficult to predict them exactly.

Earthquake load is seismic motion consists of horizontal and vertical ground motions, with the vertical motion usually having a much smaller magnitude. Further, factor of safety provided against gravity loads usually can accommodate additional forces due to vertical acceleration due to earthquakes. So, the horizontal motion of the ground causes the most significant effect on the structure by shaking the foundation back and forth. The mass of building resists this motion by setting up inertia forces throughout the structure. The magnitude of the horizontal shear force F shown in Figure 2.2 depends on the mass of the building M, the acceleration of the ground a, and the nature of the structure. If a building and the foundation were rigid, it would have the same acceleration as the ground as given by Newton's second law of motion, $F = Ma$.

However, in practice all buildings are flexible to some degree. For a structure that deforms slightly, thereby absorbing some energy, the force will be less than the product of mass and acceleration [Figure 2.2(b)]. But, a very flexible structure will be subject to a much larger force under repetitive ground motion [Figure 2.2(c)]. This shows the magnitude of the lateral force on a building is not only dependent on acceleration of the ground but it will also depend on the type of the structure.

As an inertia problem, the dynamic response of the building plays a large part in influencing and in estimating the effective loading on the structure. The earthquake load is estimated by Seismic co-efficient method or Response spectrum method. The later takes account of dynamic characteristics of structure along with ground motion.
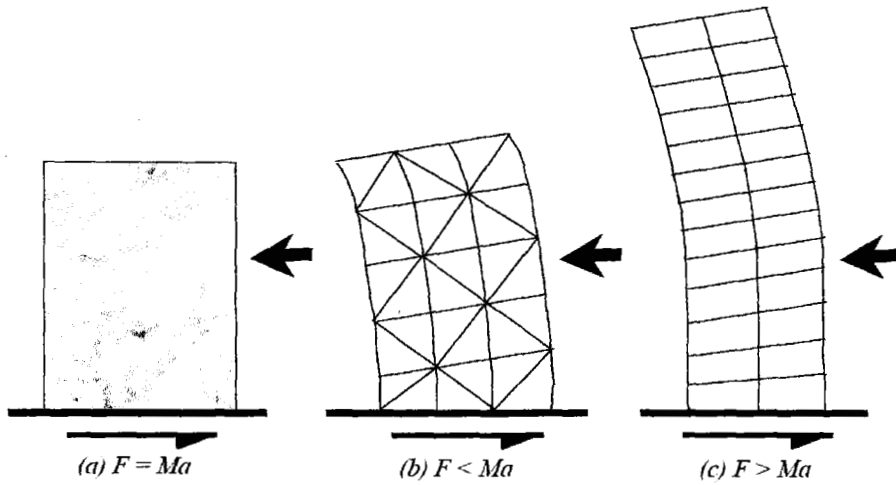
Figure 2.2: Force developed by earthquake

2.4 Lateral load resisting systems

Lateral forces due to wind or seismic loading must be considered for tall buildings along with gravity forces. Very often the design of tall buildings is governed by lateral load resistance requirement in conjunction with gravity load. High wind pressures on the sides of tall buildings produce base shear and overturning moments. These forces cause horizontal deflection in a multi-storey building. This horizontal deflection at the top of a building is called drift. The drift is measured by drift index, Δ/h, where, Δ is the horizontal deflection at top of the building and h is the height of the building. Lateral drift of a typical moment resisting frame is shown in Figure 2.3.
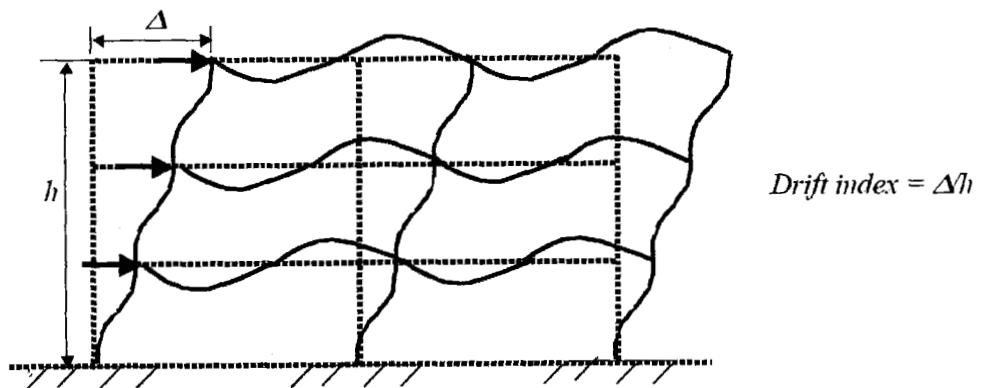

Figure 2.3: Lateral drift

A multi-storey building with no lateral bracing is shown in Figure 2.4 (a). When the beams and columns shown are connected with simple beam connections, the frame would have practically no resistance to the lateral forces and become geometrically unstable. The frame would laterally deflect as shown in Figure 2.4 (b) even under a small lateral load.
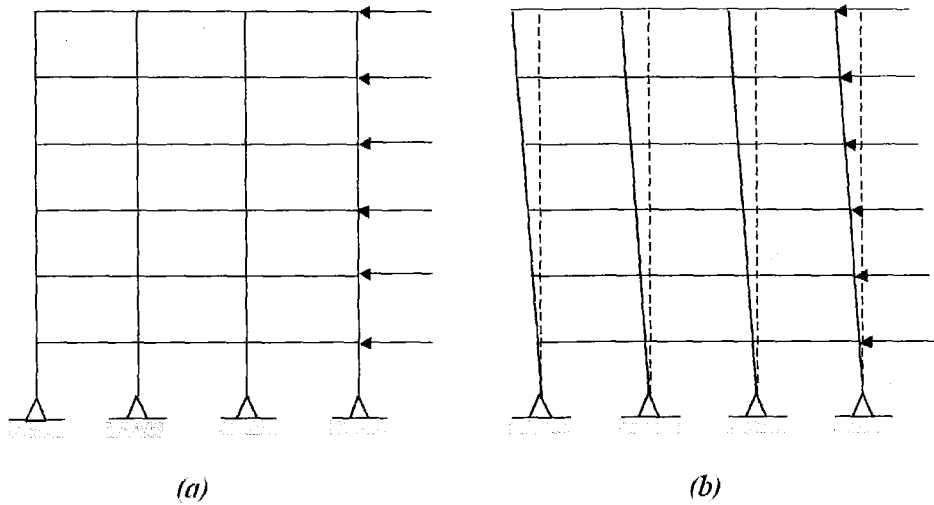

(a)   *(b)*

Figure 2.4: Multi-storey frame without lateral bracing

Rigid frames, shear walls and braced frames are commonly used to resist the lateral loads and limit the drift within acceptable range mentioned above. Combinations of these systems and certain other advanced forms are also used for very tall buildings. Rigidly jointed frames or sway-frames are those with moment resisting connections between beams and columns. It may be used economically to provide lateral load resistance for low-rise buildings. Generally, it is less stiff than other systems.
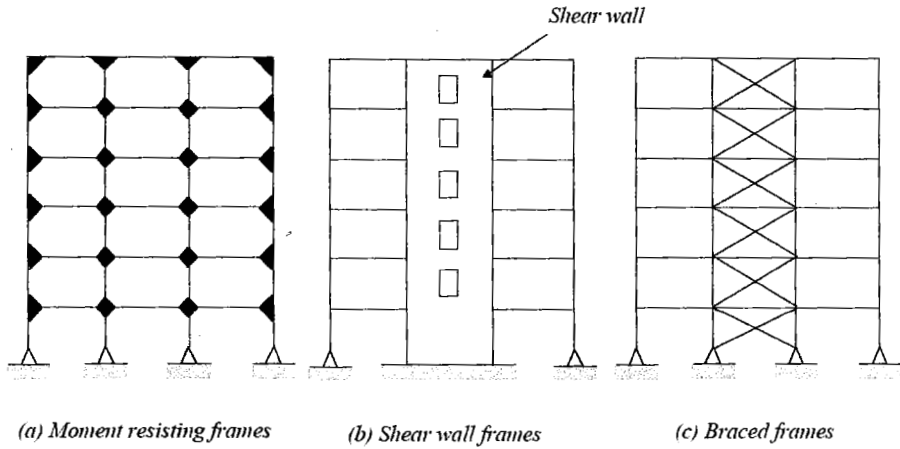
(a) Moment resisting frames *(b) Shear wall frames* *(c) Braced frames*

Figure 2.5: Lateral load resisting system

2.5 Repeated Reversed Load or cyclic load

According to Evans and Michael (2004) repeated reversed load or cyclic load indicate to periodic load which is describe according to the numerical values of the maximum or minimum amplitude with respect to zero . An element subjected to a repeated and alternating tensile and compressive stresses shown as a figure below:

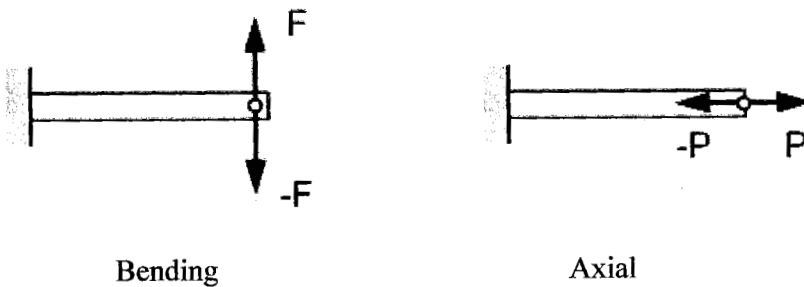
Bending   Axial

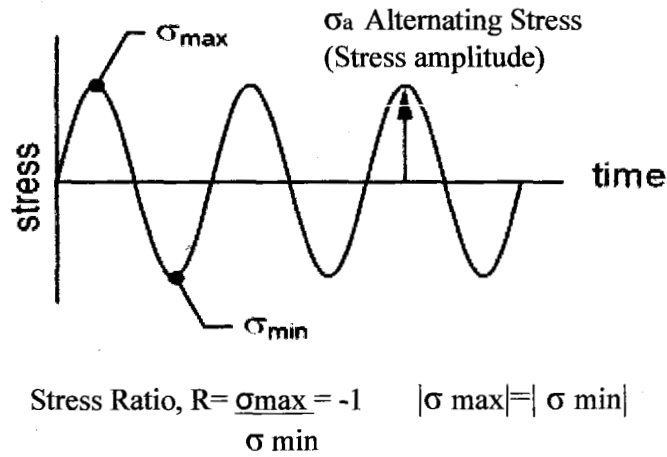Figure 2.6: Continuous total load reversal over time (Evans and Michael 2004)

Figure 2.7: The average or mean stress is zero (Evans and Michael 2004)

$$\sigma_a = \frac{\Delta\sigma}{2} = \frac{\sigma_{max} - \sigma_{min}}{2} \quad \text{=Alternating stress} \quad (1)$$

$$\sigma_m = \frac{\sigma_{max} + \sigma_{min}}{2} \quad \text{= Mean stress} \quad (2)$$

$$R = \frac{\sigma_{min}}{\sigma_{max}} \quad \text{= R value} \quad (3)$$

R = 0, repeated and one direction, i.e. stress cycles from 0 to max value.

R = -1, fully reversed

Cyclic Stress-Strain relationships can be divided to two types (Laurel Chun, Paul Knutzen, and Cynthia Shen, 2001)

I.   Cyclic Hardening

Stresses increase with increasing number of cycles

II. Cyclic Softening

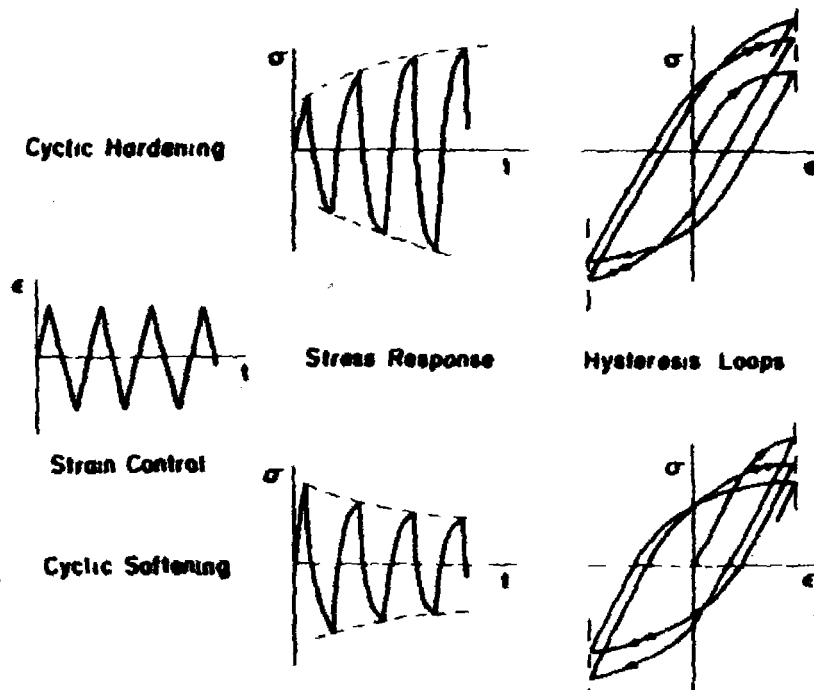Stresses decrease with increasing number of cycles

Figure 2.8: Cyclic Stress-Strain Graphs (Evans and Michael 2004)

2.6 The cyclic behavior of Reinforcing bar

According to Wei Yu (2006) on his thesis stated that the behavior of reinforced bar have been study by Johann Bauschinger (1886) which is noted that on reversal from loads above the elastic limit of a Bauschinger steel rod, a reduction in the elastic limit in the opposite direction is found. This reduction has since then called the "Bauschinger effect" and has been the subject of much research at the microscopic and macroscopic level.

If steel reinforcing bar is loaded in one direction into the inelastic range and then loading is reversed, softening of the steel resistance (yielding) will often occur before the magnitude of stress during loading in the opposite direction reaches the value of the initial yielding stress of the material. The modulus of elasticity also decreases, at a rate that is rapid after yielding but stabilizes at large strains. Tests have shown that between the maximum plastic strain and the unloading modulus a close relationship appears to exist. Moreover, the unloading modulus shows that a small recovery for reversals with a plastic strain smaller than the maximum plastic strain.

Dodd and Restrepo-Posada (1995) have found that the shape of the Bauschinger effect is also dependent on the chemical composition of reinforcing steel. Especially, the carbon content affects the shape of Bauschinger effect a lot. This will discuss in the following sections when a review of previous stress-strain models are presented.

Physically the phenomenon can be attributed to the plastic deformation that occurs in metals due to the propagation of the dislocations on slip planes most closely oriented to the maximum shear stress. As a consequence if the increased dislocation number and relative changes in the crystal lattice, some restraint tend to limit the tendency to move. To further increase plastic deformation an increased stress is required and the material strain hardens. When a dislocation reaches a new barrier in the crystal lattice it is repelled by atomic forces.

This phenomenon corresponds to the "back stress" whose macroscopic effect is the Bauschinger effect.
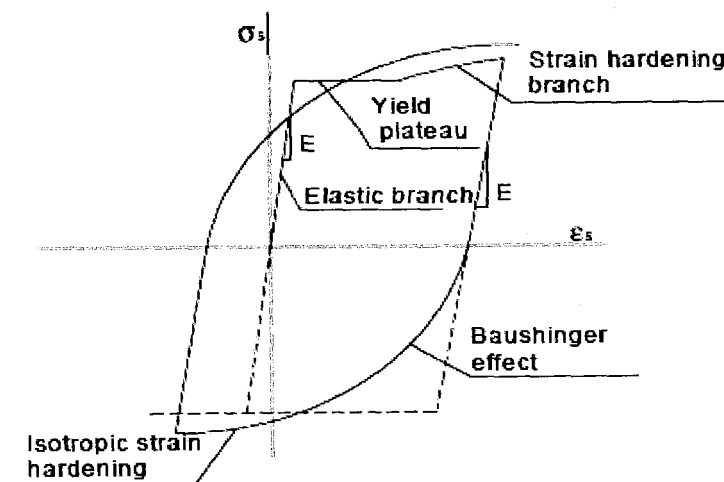
Figure 2.9: Main characteristics of stress-strain relationship of reinforcing bars under cyclic loading (Johann Bauschinger 1886)

2.7   Bond between Concrete and Reinforcing Bars

Concrete and reinforcing bar are combine to achive the best performance of structure member. According to Eligehausen et. al (1983) Bond qualities affect anchorage, lap

splices, cracking and deformations of R/C members, and it also affects the non-linear cyclic behavior of R/C beam-column joints which is complex and known to be sensitive to many factors interacting each other. In seismic design, it affects stiffness and energy dissipation; hence it is a significant property for ensuring adequate seismic performance. Under monotonic loading, bond between concrete and reinforcing bars is initially due to chemical adhesion. After a bond stress from 0.5 to 1.0 *MPa,* adhesion breaks and slip between the bars and the surrounding concrete occurs; further bond is also provided by the friction and wedging action between the cement paste and the pitting of reinforcing bars. More significantly, in deformed bars, mechanical interlock between the deformations (ribs or indentations) and surrounding concrete occurs. The interlock force eventually lead to internal bond cracks next to the deformations; at about the same time separation if concrete from the bars takes place in the region of flexural cracks. Subsequent to separation, forces from the deformations to the surrounding concrete may lead to splitting cracks, typically parallel to the bars. If these cracks can propagate without restraint, bond-splitting failure occurs. Fig. 2.10 shows a typical scale of bond response. The presence of pressure transverse to the bar (due to compressive axial loading of confinement) leads to an increase in bond resistance, mainly by suppressing early cracks.

(b) Local Bond Response Scale of the Reinforcement

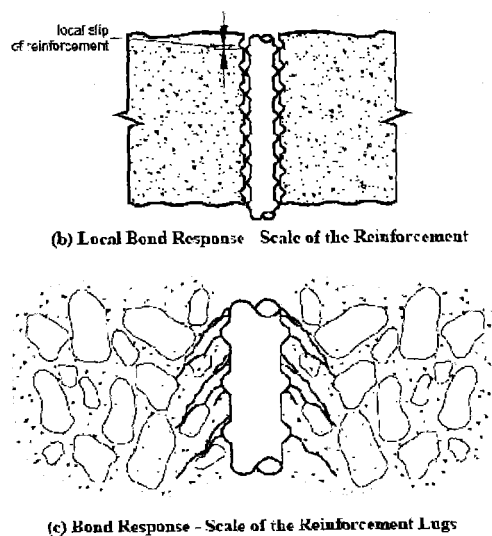**(c) Bond Response - Scale of the Reinforcement Lugs**

Figure 2.10: A typical scale of bond response (Eligehausen *et al* 1983)

During the cyclic loading at stress in excess of 70 to 80 percentage of the maximum bond stress, τ–s max , the envelop of the τ–s curve is no longer the curve corresponding monotonic loading, as shown in Fig. 2.11. The reduction in bond resistance is more pronounced as the value of slip between which cycling takes place increase, as well as the number of cycles increases.
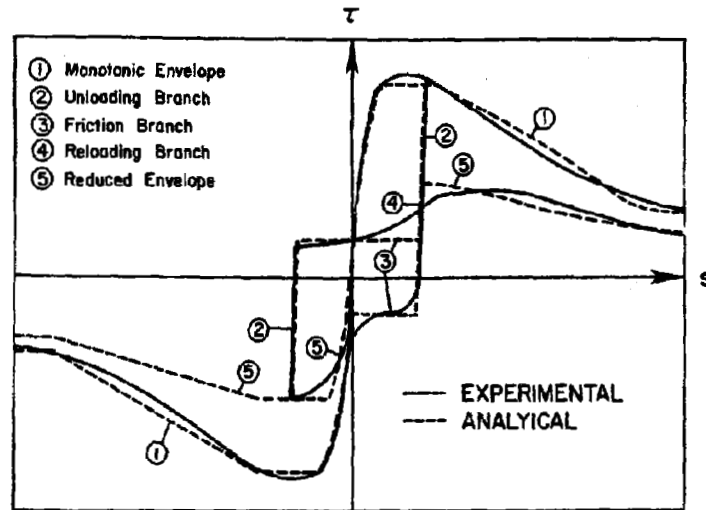

Figure 2.11: τ–s relationship under reversed cyclic loading (Eligehausen *et al* 1983)

2.8 Cyclic load behavior of exterior beam- column Joints

The internal forces acting on a reinforced concrete exterior bema-column joint under cyclic loading (or seismic action) is shown in Figure 2.12 (a). After diagonal tension cracking in the joint core, the beam and column forces are transferred across the joint acre by a diagonal compressive strut and a truss mechanism consisting of a concrete diagonal compression field and horizontal and vertical reinforcement in the joint core, as shown in Figure 2.12(b).(Wei Yu, 2006)
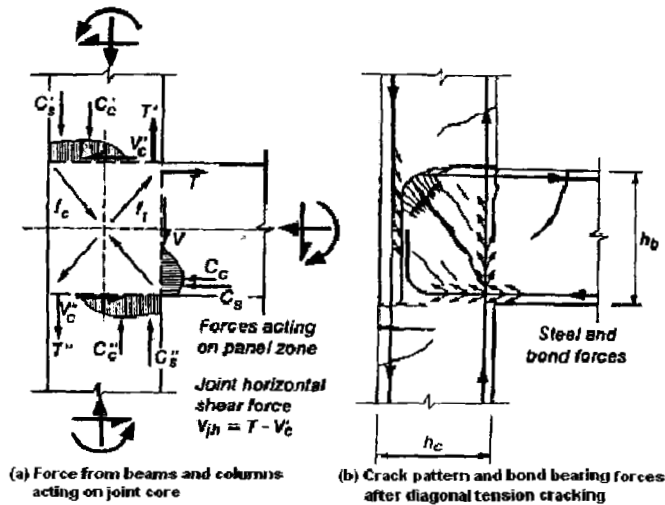
Figure 2.12: Cyclic behavior of exterior beam-column joints

Normally Repeated reverse load or cyclic load occurred under earthquake shaking. The beams adjoining a joint are subjected to moments in the same (clockwise or counterclockwise) direction (Figure 2.13). Under these moments, the top bars in the beam-column joint are pulled in one direction and the bottom ones in the opposite direction (Figure 2.14a). These forces are balanced by bond stress developed between concrete and steel in the joint region. If the column is not wide enough or if the strength of concrete in the joint is low, there is insufficient grip of concrete on the steel bars. In such circumstances, the bar slips inside the joint region, and beams loose their capacity to carry load (Murty , 2005).
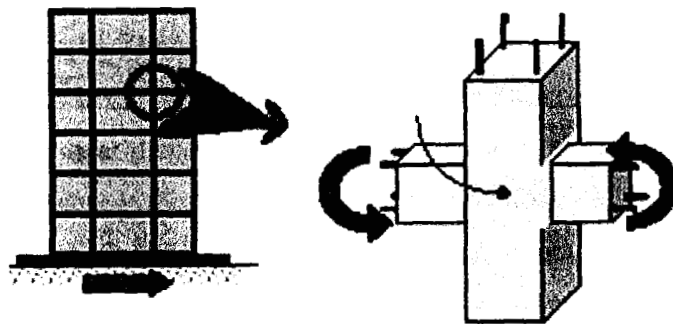

Figure 2.13: Beam-Column Joints are critical parts of a building (Murty, 2005)

Further, under the action of the above pull-push forces at top and bottom ends, joints undergo geometric distortion; one diagonal length of the joint elongates and the other compresses (*Figure* 2.14 b). If the column cross-sectional size is insufficient, the concrete in the joint develops diagonal cracks (Murty, 2005).

(a) Loss of grip on beam bars in joint region: *Large column width and good concrete help in holding the beam bars*

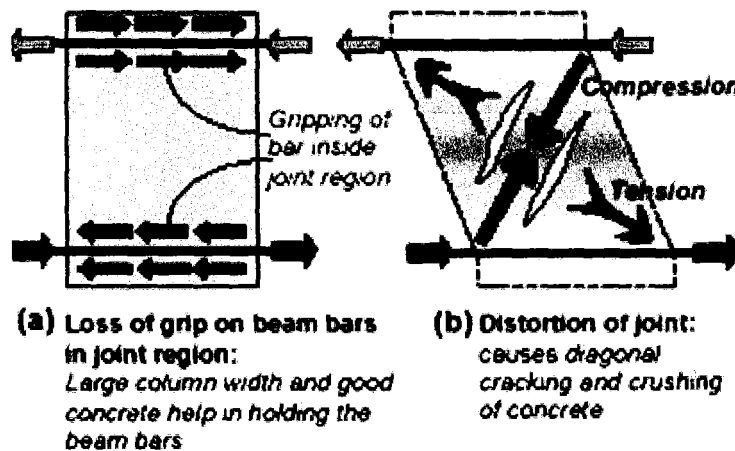**(b) Distortion of joint:** *causes diagonal cracking and crushing of concrete*

Figure 2.14: Pull-push forces on joints cause two problems– these result in irreparable damage in joints under strong seismic shaking (Murty, 2005)

2.9 Joint Mechanisms

According to Meher Prasad (2002) In the strong column-weak beam design, beams are expected to form plastic hinges at their ends and develop flexural overstrength beyond the design strength. The high internal forces developed at plastic hinges cause critical bond conditions in the longitudinal reinforcing bars passing through the joint and also impose high shear demand in the joint core. The joint behavior exhibits a complex interaction between bond and shear. The bond performance of the bars anchored in a joint affects the shear resisting mechanism to a significant extent.

The flexural forces from the beams and columns cause tension or compression forces in the longitudinal reinforcements passing through the joint. During plastic hinge formation, relatively large tensile forces are transferred through bond. When the longitudinal bars at the joint face are stressed beyond yield splitting cracks are initiated along the bar at the joint face which is referred to as 'yield penetration'.

Adequate development length for the longitudinal bar is to be ensured within the joint taking yield penetration into consideration. Therefore, the bond requirement has a direct implication on the sizes of the beams and columns framing into the joint.

In exterior joints the beam longitudinal reinforcement that frames into the column terminates within the joint core. After a few cycles of inelastic loading, the bond deterioration initiated at the column face due to yield penetration and splitting cracks, progresses towards the joint core. Repeated loading will aggravate the situation and a complete loss of bond up to the beginning of the bent portion of the bar may take place. The longitudinal reinforcement bar, if terminating straight, will get pulled out due to progressive loss of bond. The pull out failure of the longitudinal bars of the beam results in complete loss of flexural strength. This kind of failure is unacceptable at any stage. Hence, proper anchorage of the beam longitudinal reinforcement bars in the joint core is of utmost importance.

The pull out failure of bars in exterior joints can be prevented by the provision of hooks or by some positive anchorage. Hooks, as shown in Fig. 2.18 are helpful in providing adequate anchorage when furnished with sufficient horizontal development length and a tail extension. Because of the likelihood of yield penetration into the joint core, the development length is to be considered effective from the critical section beyond the zone of yield penetration. Thus, the size of the member should accommodate the development length considering the possibility of yield penetration. When the reinforcement is subjected to compression, the tail end of hooks is not generally helpful to cater to the requirements of development length in compression. However, the horizontal ties in the form of transverse reinforcement in the joint provide effective restraints against the hook when the beam bar is in compression.

2.10   Joint Failure

The moment resisting frame is expected to obtain ductility and energy dissipating capacity from flexural yield mechanism at the plastic hinges. Beam column joint behaviour is controlled by bond and shear failure mechanisms, which are weak sources for energy dissipation. The joint should have sufficient strength to enable the maximum capacities to be mobilized in the adjoining flexural members and the

degradation of joints should be so limited such that the capacity of the column is not affected in carrying its design loads.

Joint zone is one of the weak and critical sections in one structure system (Park and Paulay, 1975). There are five types of failure that can occur at the beam column joint (Meinheit and Jirsa, 1981). The first type represents connection failure by beam hinging as shown in Figure 2.15(a). This failure happens due to the formation of plastic hinges at the end of the beam in joint zone. This condition occurs when the beam cannot resist higher load and the reinforcement failed with the development of a lot of cracks. Although the joint zone can still resist the load; the beam failure will result in the failure of the joint core. The second type of failure is used to represent column-hinging failure as shown in Figure 2.15(b). This failure happens when the plastic hinges occur at column, either caused by shear force action or compression force. As in the case of beam failure, a lot of cracks also can be seen in column failure, which means that the column reinforcement cannot resist loading. These kinds of failure have to be avoided because this condition can caused the frame to sway and hard to be repaired. The third failure type is caused by the spalling of concrete cover at joint zone as shown in Figure 2.15(c). This happen because of cracks that occur at the joint face, where the crack concrete burst when the load increased. The spall-of concrete cover must be avoided because this can cause the decreasing of the compressive load in the column. The fourth failure type, which is failure of anchorage bar inside the joint (Figure 2.15(d)). This failure consists of exterior column joint. Reinforcements that have to resist negative moment in the beam must be anchored into column as anchorage length. Inadequate anchorage length or poor detailing will cause this failure. Small radius of bend bars will produce high bearing stresses and contribute to anchorage failure. The frame structure cannot transfer the biaxial shear and decreased the structure capacity in absorb energy. The fifth failure type that is joint shear failure (Figure 2.15(e)). Loading at the beam will cause the horizontal shear force in column especially in the exterior beam-column joint. The combination of shear stresses and tensile forces also compressive forces at the beam reinforcement and column produced tensile stress and compression stress in the joint zone. The value of tensile stress may be greater when the joint segments reached the ultimate capacity limitation, and this will cause cracks at the joint zone. This failure almost same as anchorage bar failure, where the frame structure cannot transfer the biaxial shear and decreased the structure capacity to absorb energy.
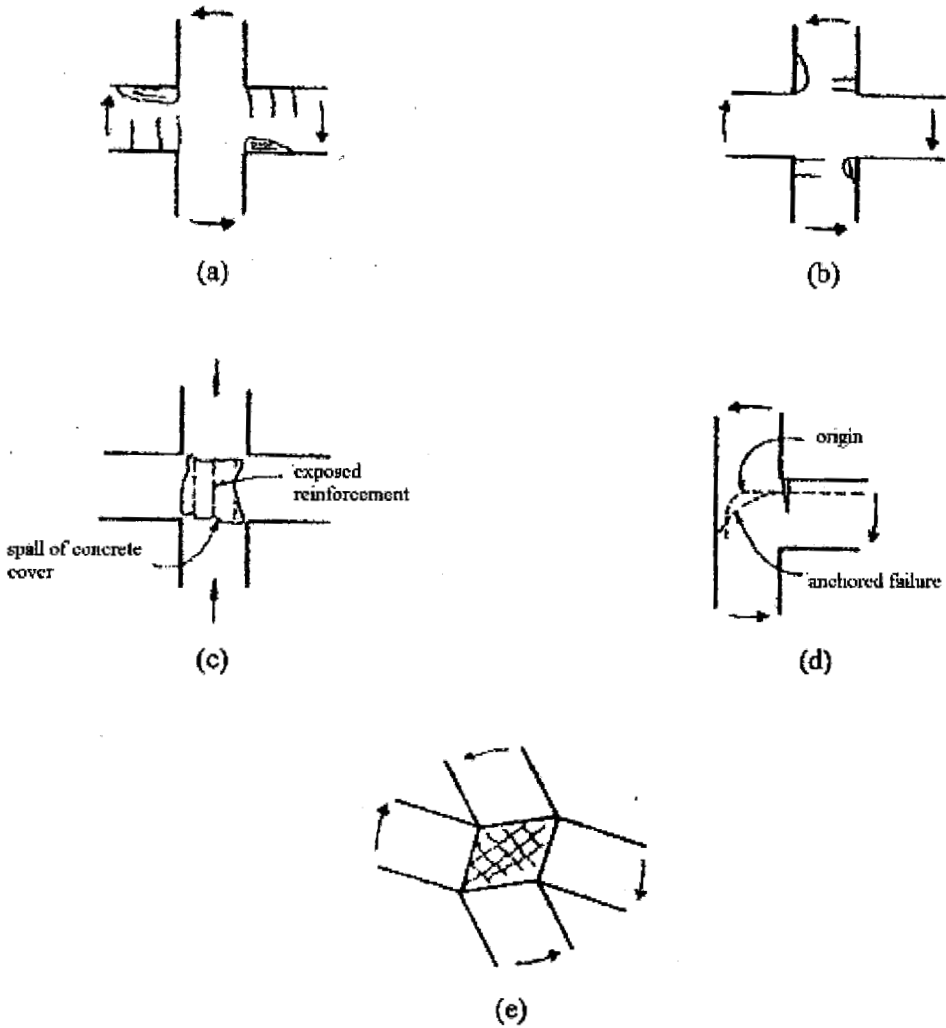
Figure 2.15: Type of failure mode in connection zone (a) Beam hinging failure, (b) Column hinging failure, (c) Failure by spall of concrete cover, (d) Anchorage failure, and (e) Joint shear failure (Meinheit and Jirsa, 1981)

2.11 Factors Affecting the Joint Behaviour

From the previous study, many researchers have found some factors that affect the joint behaviour as explained below.

2.11.1 Concrete Strength

Many researchers conducted laboratory tests on beam-column specimens to investigate the effectiveness of different concrete strengths at column and beam or slab on the behaviour of beam-column joints. According to Marzouk *et. al.* (1996) tested seven specimens of internal beam-column joints with the concrete strength in the column higher than the concrete strength in beam. The results show that the column with higher strength concrete compared to normal concrete strength for column has increased the joint shear strengths for the small moment value by about 5% and about 17 % for high moment value. They also conducted the use of higher concrete strength in column that affected in the joint shear stress, as well as on the behaviour of beam-column joint. The use of higher strength concrete in column shows better stress distribution between column and slab. Existing research (Bianchini et al, 1960; Gamble and Klinar, 1991; Ospina and Alexander, 1997; Ospina and Alexander, 1998) shows that the effective concrete strength of high strength concrete column with an intervening normal strength concrete floor is generally greater than the concrete strength of the floor but less than that of the column. For a given combination of column and floor concrete, the effective strength of the joint depends on the degree to which the surrounding floor confines the joint. According to Juliana Jopely (2005), eleven specimen of external beamcolumn joints were tested using Grade C30 in the beam and C60 in the column and connection zone, the results show that the joint shear strength of external beam column joints cast with Grade C30 were 5% lower than that cast with Grade C30 concrete.

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