Seismic Assessment and Retrofit of Reinforced Concrete Columns

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By

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Cambridge Scholars Publishing

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To My Dedicated Parents and Brother

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Onassis
Foundation

CHAPTER ONE

INTRODUCTION

Existing reinforced concrete buildings constructed before the development of modern seismic design provisions represent one of the largest seismic safety concerns worldwide. Such buildings are vulnerable to significant damage and even collapse when subjected to strong ground shaking. The collapse of reinforced concrete buildings has been the cause of many of the fatalities in past earthquakes. Since 1980, after the capacity design concept was introduced into the seismic design code provisions, the seismic safety gap between the newly designed seismic resistant buildings and those constructed before 1980 has widened, causing worldwide concern. The crucial issue that was evident after the earthquakes in 1999 in Athens (Partnitha) and in Turkey (Kocaeli) and was underlined by the destructive earthquake of L'Aquila (2009) in Italy (an event which the author experienced personally as a resident of L'Aquila at the time) is the need to improve assessment and retrofit procedures for existing reinforced concrete buildings.

Reinforced concrete (RC) columns play a very important role in structural performance. Behaviour of RC columns in shear and flexure has been studied for decades. In the case of flexural behavior, sectional analysis, or a fiber model in one-dimensional stress field gives acceptable predictions in terms of ultimate strength and yielding deformation. Performance of reinforced concrete columns dominated by shear or shear-flexure cannot be estimated by applying only a sectional analysis because shear behavior is not taken into account in the approach. For evaluating the shear response of structural elements, such as beams and columns, many analytical models and theories have been presented in the past. Some of the most commonly used approaches are strut and tie models (Mörsh 1902, Ritter 1899) and the Modified Compression Field Theory (MCFT) (Vecchio & Collins 1986). MCFT is a powerful tool to model the response of RC elements subjected to in-plane shear and normal stresses. The method is based upon a large number of membrane elements tests and treats reinforced concrete in an average way. Specifically, the method is formulated in terms of average stresses and strains across the element and is supplemented by local crack checks. The method is formulated with consideration to equilibrium, compatibility, and approximate stress-strain relationships of the materials.

Recently, another aspect that has roused the interest of researchers is the axial failure of columns that can lead to collapse of a building (Elwood and Moehle 2005). Before the introduction of special requirements in the 1970s, reinforced concrete building frames constructed in zones of high seismicity had details and proportions similar to frames designed primarily for gravity loads. Columns generally were not designed to have strengths exceeding beam strengths, so column failure mechanisms often prevail in buildings dating from that era. Relatively wide spacing of transverse reinforcement was common, such that column failures may involve some form of shear or combined flexure – shear failure. As shear failure proceeds, degradation of the concrete core may lead to loss of axial load carrying capacity of the column. As the axial capacity diminishes, the gravity loads carried by the column must be transferred into neighboring elements. A rapid loss of axial capacity will result in the dynamic redistribution of internal actions within the building frame and may progressively lead to collapse.

During earthquake excitation columns can experience a wide variety of loading histories, which may consist of a single large pulse or several smaller-amplitude cycles, occasionally leading to either shear failure or even collapse – i.e. a loss of gravity-load bearing capacity of the column. Previous research has demonstrated that the onset of this type of collapse cannot be quantified unilaterally by a single combination of shear force and axial load values, but rather, it is characterized by an interaction envelope that depends on the history of loading and the peak magnitude of deformation exertion attained by the column (max. drift demand). Recent studies (Chapter 2) attribute particular influence to the final mode and characteristics of failure by the occurrence of fluctuating axial load about a mean value, on some occasions the load becoming actually tensile due to the overturning effects imparted by the earthquake. Furthermore it has been demonstrated that an increase in the number of cycles past the yield displacement can result in a decrease in the drift capacity at shear failure. Understanding these effects and developing mechanistic tools by which to identify the characteristics of failure at the loss of axial load bearing capacity and the implications of drift history is one of the objectives of this book.

In the present book a fiber beam-column element accounting for shear effects and the effect of tension stiffening through reinforcement-to-concrete bond was developed, in order to provide an analytical test-bed for simulation and improved understanding of experimental cases where the testing of RC columns actually led to collapse. Emphasis is particularly laid on lightly reinforced columns. The combined experimental/numerical results provided useful information for the definition of plastic hinge length in columns through consideration of yield penetration effects. The required confined zone in critical regions of columns and piers undergoing lateral sway during earthquakes is related to the plastic hinge length where inelastic deformation and damage develops. The exact definition of the plastic hinge length stumbles upon several uncertainties, the most critical being that the extent of the inelastic region evolves and spreads with the intensity of lateral displacements. Design codes quantify a reference value for the plastic hinge length, through calibrated empirical relationships that account primarily for the length of the shear span and the diameter of primary reinforcing bars. The latter term reflects the effects of bar yielding penetration in the support of columns. Here a consistent definition of plastic hinge length is pursued analytically with reference to the actual strain state of the reinforcement.

Over the past three decades, fibre-reinforced polymer (FRP) composites have emerged as an attractive construction material for civil infrastructure, rehabilitation, and renewal. These advanced materials have been successfully used for reinforcing new structures as well as the strengthening/rehabilitation of existing buildings and bridges. The use of FRP composites, analysis and design, and techniques for installation are continually being researched and it is anticipated that the use of these advanced materials will continue to grow to meet the demands of the construction industry. Recent seismic events around the world continue to underline the importance of seismic retrofit and strengthening of existing concrete structures leading to the need for new, practical, occupant-friendly and cost-effective remedial solutions.

In this context, the Fiber Reinforced Polymer (FRP)-confined concrete model contained in a well-known Bulletin by the International Federation for Structural Concrete (*fib*) has been enhanced to take into account the superposition of the confining effects of the already existing steel reinforcement with that of the FRP jacketing applied when retrofitting RC columns. Columns are here modeled with a fiber-based nonlinear beam-column element (with displacement formulation) in which the constitutive law for concrete presented in this book is implemented. This allows for the immediate incorporation of shear strains (uncoupled from the normal ones) at the material level. The averaged response of the two different regions—concrete core and concrete cover—in the cross-section allows the assignment of a unique stress-strain law for all the fibers/layers of the section. Correlation with experimental studies from the literature is performed to validate the proposed iterative procedure.

Specifically, the organization of the present book is the following: After the introduction in Chapter 1, Chapter 2 contains a literature review of the part of seismic assessment of old-type RC columns. Chapter 3 presents the correlation of - the state of the art – analytical models for seismic assessment of reinforced concrete columns with the experimental results of a well-known experimental database. Chapter 4 defines the plastic hinge length in columns though consideration of yield penetration effects. A mechanically consistent approach in determining inelastic rotation capacity of reinforced concrete columns is introduced. Chapter 5 presents the development of a force-based fiber beam-column element accounting for shear and tension stiffening effects. Chapter 6 presents new developments on FRP seismic retrofit of RC columns with confining wraps or jackets that has proven to be an efficient technique for the seismic retrofit of structures. A new constitutive model for FRP -and steel-confined concrete, including shear effects, is included in this Chapter. Finally, in Chapter 7 important conclusions based on the described research in this book are drawn.

To sum up, this book is introducing recent advances in research that intends to attract academic staff, researchers, under- or post- graduate students and professional engineers dealing with seismic assessment, repair and retrofit of RC structures such as buildings and bridges.

CHAPTER TWO

STATE OF THE ART ON SEISMIC ASSESSMENT OF REINFORCED CONCRETE COLUMNS

The procedure of estimating the strength, the deformation capacity and the expected mode of failure in primary members of a RC frame structure, that is, the complete process of seismic assessment, has been recently supported by background documents in both Europe and the U.S. (KAN.EPE. 2014, EN 1998-3 2005, ASCE/SEI-41 2007, and most recently by the draft of the New Model Code 2010 by the *fib*). The acceptance criteria proposed provide a complex system of evaluation, but the various steps of this process are not vested with a uniform level of confidence as compared with the experimental results. Strength values can be estimated with sufficient accuracy only if the modes of failure involved are ductile. The level of accuracy is degraded when considering brittle mechanisms of resistance, and the associated deformation capacities, which are used as a basis for comparison with deformation demands to assess the level of performance (i.e. the damage), generally do not correlate well with proposed Code estimations. However, in the process of assessment it is a critical matter seriously affecting public safety, to determine whether flexural yielding will precede shear failure (so as to ensure ductility) or whether a brittle failure ought to be anticipated prior to reinforcement yielding. Even when flexural yielding may be supported it is also important to dependably estimate the ductility level beyond which shear strength may be assumed to have degraded below the flexural strength, leading to a secondary post-yielding failure that limits the available deformation capacity (Fig. 2-1).

Stiffness properties and inelastic the earthquake response of frame members are usually studied based on a statically determinant structure comprising a cantilever reinforced concrete column under lateral loading. Given the material properties (be they nominal, assumed or experimentally measured), geometry, the loading conditions and loading history, it is theoretically possible to analyze the cantilever so as to study the interactions between various aspects of its response such as flexure, shear and reinforcement development capacity. In recent years the fixed ended column specimen in lab experiments is preferred to be compared to the cantilever arrangement, since the interaction of two end moments and more realistic curvatures can be obtained, whereas they are more versatile in dynamic tests (as it is possible to mount masses on top of the restraining beam at the upper end of the column, thus simulating more realistically the actual circumstances in the field). Moreover, in the case of lightly reinforced concrete columns which are representative of older construction, major inclined shear cracks have been seem to occur in the midheight column region (near the point of column inflection), a crack pattern that cannot be reproduced with the cantilever specimen since its tip is free to rotate (only restrained in translation) and sustains no damage in that region. In addition the elongation due to damage of the double curvature member is more representative of a typical building column under earthquake loading. The assessment performance objectives in such experiments can be categorized and documented by obtaining the full inelastic response until the collapse of the RC column.

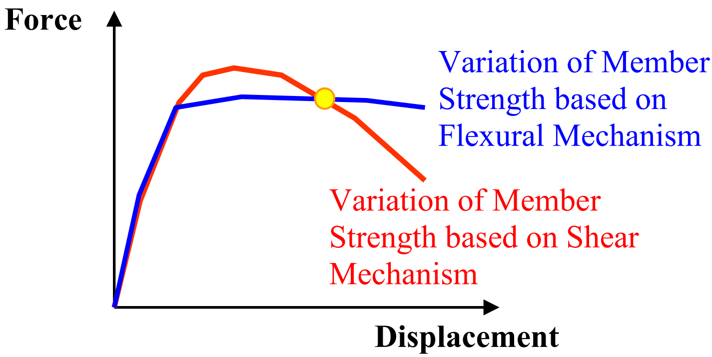

Fig. 2-1: Capacity curve due to flexural or shear mechanism.
Failure denoted with yellow point.

According to Eurocode 8, Part 3 (EN 1998-3, 2005), the fundamental performance criteria related to the state of the structural damage are defined through three Limit states that span the range of the member resistance curve (Fig. 2-2.a), and are defined according to the severity of damage that they represent as follows: “Damage Limitation (DL)”, “Significant Damage (SD)”, and “Near Collapse (NC)”. The target displacement of the column based on the earthquake load defines which of these Limit States are reached. In the following figure (Fig 2-2.b) the performance objectives for these Limit States are documented in practical terms.

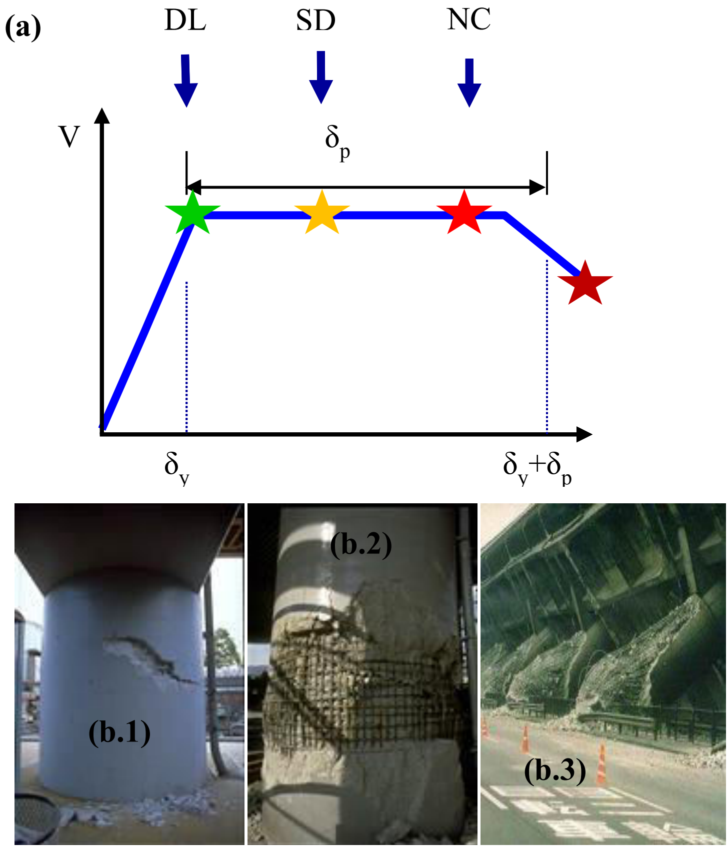

Fig.2-2: Damage of bridge columns: a) Member resistance curve and definition of limit states according with EN 1998-3 (2005).
(b.1) Damage Limitation Limit State (b.2) Significant Damage Limit State (b.3) Near Collapse Limit State.

The objective of this chapter is to critically review and identify, through a thorough review of the published experimental evidence, the critical issues affecting the resistance curve of columns during earthquake action (strength and deformation capacity) and the limiting brittle modes of failure. This is important since the column resistance curve eventually controls the buildings' resistance in a relatively straightforward manner (Fotopoulou et al., 2011) whereas a sudden loss of column strength to overbearing loads may lead to collapse and human losses. In the context of a displacement based evaluation framework, not only the relevant shear strength is important, but also the corresponding column displacement capacity. In this regard, recent experimental evidence of shear critical reinforced concrete columns will be reviewed along with recently developed analytical models and the relevant state of the art of code assessment procedures.

Existing Experimental Studies on Shear Dominated RC Columns

The behaviour of shear-critical reinforced concrete columns has been the subject of extensive study and research in recent years as this seems to remain a challenging concrete mechanics problem. Shear dominant behaviour is reported in columns with a low aspect ratio, but also in lightly reinforced columns containing low ratios of transverse reinforcement. Section geometry (rectangular or circular sections) is one of the parameters that differentiate the available test results; cyclic pseudo-static, hybrid pseudo-dynamic and dynamic tests are included in the review. Some experimental studies are dedicated to the influence of axial load fluctuation on the response of the column (fluctuation of axial load about the value that is affected by the overbearing loads occurring during the seismic event as a result of the overturning action of lateral loads, and is most significant in columns located at a distance from the centre of mass of the building, i.e., on the perimeter of the structure).

The same effect is seen in bridge piers belonging to multiple-column bents where it may be easily demonstrated that the axial load fluctuation is proportional to the horizontal (seismic) forces. Columns are also subjected to the vertical components of ground motion, which is not correlated concurrently with the horizontal loading. Past earthquake records have shown that in some cases, vertical ground motions cannot be ignored, particularly for near-fault situations. For example, the lateral displacement ductility of a column, designed based on constant axial load with a relatively low axial load ratio, can become unsatisfactory when the actual axial load due to the overturning effects or where the vertical ground motion exceeds the "balanced" axial load limit (i.e., about 40% of the column crushing load). The problem becomes even more significant when shear design is considered. The increase of axial load from the design level (which typically is in the order of 5% to 10% of the crushing load) to the level of the balanced value generally increases the column flexural capacity causing a commensurate increase in the design shear demand (based on capacity design principles). On the other hand, a change in the axial load value from compression to tension may compromise significantly the column shear strength.

A review of Influential Cyclic Column Tests

From among the multitude of published tests on cyclically loaded columns under lateral displacement reversals (see also Chapter 3), a number of tests have received greater attention as their response was used as points of reference in calibrating the design expressions for shear published in the literature. On account of the weighty contribution of these experimental studies to the formation of the current assessment framework, these studies are reviewed separately in the present work.

Ang, Priestley and Paulay (1989) performed experimental tests to study the seismic shear strength of circular columns. A series of twenty-five 400 mm-diameter columns, considered to be approximately one-third scale models of typical bridge columns, were constructed and tested under cyclic reversals of lateral loading, as part of a major investigation into the strength and ductility of bridge pier columns. Variables in the test program included axial load level, longitudinal reinforcement ratio, transverse reinforcement ratio and aspect ratio. The column units were tested as simple vertical cantilevers. Results indicated that the shear strength was dependent on the axial load level, the column aspect ratio, the amount of transverse spiral reinforcement and the flexural ductility displacement factor. At low flexural ductilities, the additive principle for shear strength, based on a concrete contribution plus a 45-deg truss mechanism involving the spiral reinforcement and diagonal concrete compression struts, described the behavior quite well. But at flexural displacement ductilities greater than two, the tests demonstrated a gradual reduction of lateral load strength with increasing ductility, whereas the inclination of the diagonal compression struts of the truss mechanism relative to the longitudinal axis decreased. Here it is worth noting that significant rotations occurred at the base of these specimens artificially distorting the data in the direction of more excessive strength loss due to P-Δ effects (Ioannou and Pantazopoulou, 2016).

Wong, Paulay and Priestley (1993) conducted a series of biaxial tests that included 16 circular (400 mm-diameter) reinforced concrete cantilever columns with an aspect ratio of two and different spiral reinforcement contents in order to investigate the sensitivity of the strength and stiffness of shear-resisting mechanisms to various displacement pattern and axial compression load intensities. Elastic shear deformations in squat circular columns with small or no axial compression load were found to be significant. It was concluded that shear deformation ought to be included explicitly in the estimation of initial stiffness of a column, so that a reliable relation between the ductility demand and the corresponding drift could be established. A general observation was that in comparison with uniaxial displacement paths, biaxial displacements led to more severe degradation of stiffness and strength, and thereby, increased energy dissipation. However, the reduction of initial shear strength and ductility capacity of squat columns (recall that the aspect ratio of the tested columns was equal to 2), subjected to biaxial displacement history was not very significant. The value of the dependable displacement ductility level attained during biaxial displacements was, on average, less (i.e. a value difference of 1) than that obtained in identical units subjected to uniaxial loading history. Initial shear strength of units with brittle shear failure was reduced by about 5 to 10 percent, depending on the axial load level when biaxial rather than uniaxial loading was considered. Finally, one more important finding was that the shear carried by spirals was underestimated when using a 45-deg potential failure plane; the observed major diagonal cracks developed in squat columns at much lower angles with respect to the longitudinal axis of the member.

Lynn et al. (1996) constructed and tested 8 full-scale square section (457 mm) columns that had widely-spaced perimeter hoops with 90-degree bends with or without intermediate S-hooks and with longitudinal reinforcement with or without short lap-splices. The columns had an aspect ratio of 3 and were loaded with constant axial load at low and intermediate levels, and were subjected to lateral deformation cycles until the column was incapable of supporting a lateral or vertical load. Failure modes included localized crushing of concrete, reinforcement buckling, lap-splice and flexural bond splitting, shear and axial load collapse. Loss of gravity load capacity occurred at or after significant loss of lateral force resistance. Where response was governed by a shear, gravity load failure occurred soon after loss of lateral force resistance. Where response was initially governed by lap-splice deterioration and gravity loads were relatively low, gravity load resistance was maintained until eventual shear failure occurred. Where response was predominantly flexural, gravity load capacity was maintained to relatively large displacements.

As earthquakes and laboratory experience show that columns with inadequate transverse reinforcement are vulnerable to damage including shear and axial load failure, another study in this direction is by Sezen and Moehle (2006). The latter included four full-scale square section (457 mm) columns (aspect ratio equal to 3) with light transverse reinforcement that were tested quasi statically under unidirectional lateral loads with either constant or varying axial loads. Test results showed that responses of columns with nominally identical properties varied considerably depending on the magnitude and history of axial and lateral loads applied. For the column with a light axial load and reversed cyclic lateral loads (applied through a displacement history), apparent strength degradation triggered shear failure after the flexural strength was reached. Axial load failure did not occur until displacements had increased substantially beyond this point. The column with high axial load sustained brittle shear compression failure and lost axial load capacity immediately after shear failure, pointing out the necessity of seismic evaluations to distinguish between columns on the basis of axial load level. The column tested under varying axial load showed different behavior in tension and compression, with failure occurring under compressive loading.

A review of relevant Pseudodynamic Tests

It was stated earlier that columns in RC structures carry axial forces owing to dead and live loads and a combined varying axial force, moment and shear when excited by earthquake ground shaking. The varying axial loads lead to simultaneous changes in the balance between their supply and demand in axial, moment and shear to an extent that eludes adequate estimation by the code models. To consider the time varying effects of the ground motion on these combined actions, simulated dynamic loads were applied using a hybrid simulation of the earthquake effects on the structural model wherein the column specimen is assumed to belong. Kim et al. (2011) used hybrid simulation, where an experimental pier specimen was tested simultaneously and interactively with an analytical bridge model which was modelled on the computer; at each step of the dynamic test the forces applied on the specimen were calculated by solving the dynamic equation of motion for the structure where the stiffness contribution of the modelled column in the global structural stiffness was estimated from the measured resistance in the previous step. Additionally, two cyclic static tests with constant axial tension and compression were performed to study the effect of the axial load level on the bridge piers. It was found that by including vertical ground motion the axial force fluctuation on the test specimen increased by 100%, resulting at times in a net axial tension that was not observed under horizontal motion alone.

This high axial force variation led to a fluctuation of lateral stiffness and a more severe outcome of cracking and damage. Moreover, inclusion of vertical ground motion significantly affected the confining spiral strains. Thus, whereas the maximum spiral strain of the specimen subjected to horizontal ground motion occurred at 20% of the pier height, in the case of an identical specimen subjected to combined horizontal and vertical excitations it occurred at 55% of the pier height. Thus, it was estimated that the spiral strain increased by 200% when vertical ground motion was included. Therefore, in this example, the deterioration of shear capacity due to vertical ground motion was experimentally demonstrated. Also, whereas the test specimen that was subjected to constant axial compression experienced brittle shear failure including rupture of the spiral reinforcement, the companion specimen that was subjected to moderate tension showed ductile behavior. Comparing the strength at the first peak of displacement, it was found that the lateral load strength of a specimen with constant axial tension increased marginally with increasing displacement; the response of the specimen with axial compression showed significant strength degradation. Hence, considering observations from the two tests described above, it was clearly shown that different axial load levels influence the pier behavior significantly and can ultimately dictate the failure mode.

Shake Table Tests conducted on Columns

Shake table tests were designed by Elwood (2002) to observe the process of dynamic shear and axial load failures in reinforced concrete columns when an alternative load path is provided for load redistribution. The test specimens were composed of three columns fixed at their bases and interconnected by a beam at the upper level. The central square section column had a wide spacing of transverse reinforcement rendering it vulnerable to shear failure and subsequent axial load failure during testing. As the central column failed, the shear and axial loads were redistributed to the adjacent ductile circular columns. Two test specimens were constructed and tested. The first specimen supported a mass that produced column axial load stresses roughly equivalent to those expected for a seven-story building. In the second specimen hydraulic jacks were added to increase the axial load carried by the central column, thereby amplifying the demands for redistribution of the axial load when the central column began to fail. Both specimens were subjected to one horizontal component of a scaled ground motion recorded during the 1985 earthquake in Chile. A comparison of the results from the two specimens indicates that the behavior of the frame is dependent on the initial axial stress on the center column. The specimen with a lower axial load failed in shear- but maintained most of its initial axial load. For the specimen with a higher axial load, shear failure of the center column occurred at lower drifts and earlier in the ground motion record, and was followed by axial failure of the center column. Displacement data from immediately after the onset of axial failure suggest that there are two mechanisms by which the center column shortens during axial failure: first, by large pulses that cause a sudden increase in vertical displacement after a critical drift is attained; and second, by smaller oscillations that appear to 'grind down' the shear-failure plane. Dynamic amplification of axial loads transferred from the center column to the outside columns was observed during axial failure of the center column.

An additional study by Ghannoum and Moehle (2012) includes earthquake simulation tests of a one-third-scale, three-storey, three-bay, planar reinforced concrete frame which was conducted to gain insight into the dynamic collapse of older-type construction. Collapse of the frame was the result of shear and axial failures of columns with widely spaced transverse reinforcement. The frame geometry enabled the observation of the complex interactions among the failing columns and the surrounding frame. The tests showed that the failure type and rate depended on the axial load level, stiffness of the surrounding framing, and intensity and duration of shaking. Column shear and axial behavior, including strength degradation, was affected by both large lateral deformation excursions and cycling at lower deformations. Low-cycle fatigue caused column collapse at significantly lower drifts than anticipated. It was concluded that current models and standards for estimating the shear and axial failure of columns do not account for low-cycle fatigue and can be unconservative, particularly for columns subjected to long-duration seismic motions. Moreover, models for shear strength degradation of reinforced concrete columns should account for both deformation and cyclically-driven damage. Finally, it was seen that structural framing surrounding the failing columns enabled vertical and lateral force redistribution that delayed or slowed down progressive structural collapse.

Code Criteria for Shear Strength Assessment of RC Columns

Behavior of reinforced concrete columns in combined shear and flexure has been studied extensively (see also Chapter 3). In the case of flexural behavior, sectional analysis, or a fiber model considering normal stresses provides acceptable estimations in terms of ultimate strength and yielding deformation. Performance of reinforced concrete columns dominated by shear or shear-flexure cannot be estimated by applying only sectional analysis because shear behavior concerns the member and not a single cross section. In these cases it is necessary to couple a shear strength model with the flexural model – and by considering independently the degradation of each with increasing deformation, to determine the prevailing mechanism that controls the mode of failure of the member at the reference performance limit. Several code assessment procedures define the shear strength and its rate of degradation with increasing displacement ductility by evaluating the concrete contribution and the transverse steel reinforcement contribution to shear strength. Actually the existing code methodologies are differentiated regarding the concrete contribution term whereas the truss analogy for steel contribution is adopted almost universally in all proposals with a minor point of discussion being the angle inclination of the primary shear crack of the column that activates the steel stirrups contribution (Fig. 2-3). The various aspects of the code assessment of shear strength will be covered in the following sections.

It is generally acknowledged that shear failure of RC structures signifies rapid strength degradation and significant loss of energy dissipation capacity. Reconnaissance reports from past strong earthquakes highlight the susceptibility of RC column webs to diagonal tension cracking that frequently leads to a brittle shear failure. Shear strength degradation ensues after the opening of the diagonal cracks which eliminate the mechanism of force transfer via aggregate interlock. To avoid shear failure, shear strength should exceed the demand corresponding to attainment of flexural strength by a safety margin.

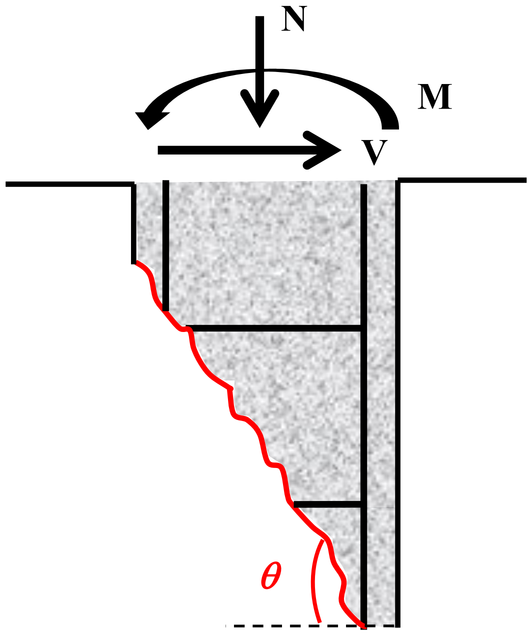

Figure 2-3: Angle inclination of the primary shear crack.

For the mechanics of shears in reinforced concrete, most issues relating to physical interpretation are still fraught with considerable debate. For example, consensus is lacking as to the physical significance of the concrete contribution term and to mathematical description of tension-based sources of shear-strength and their relationship to strain intensity and cyclic displacement history. According to EN 1998-3 (2005), the cyclic shear resistance, V_R, decreases with the plastic part of ductility demand, expressed in terms of ductility ratio of the transverse deflection of the shear span (Fig 2.4) or of the chord rotation (Fig. 2.4) at member end: $\mu_\Delta{}^{pl} = \mu_\Delta - 1$. For this purpose $\mu_\Delta{}^{pl}$ may be calculated as the ratio of the plastic part of the chord rotation, θ_p, normalized to the chord rotation at yielding, θ_y.

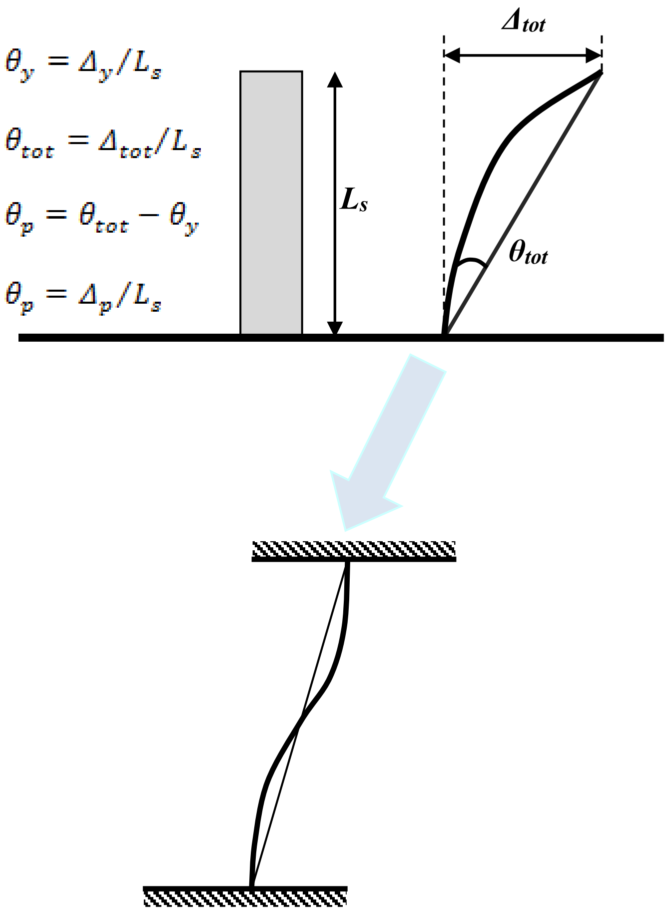

Figure 2-4: Definition of chord rotation of a cantilever reinforced concrete column (top) modeling the shear span of an actual column (bottom).

Thus, EN 1998-3 (2005) defines shear strength accounting for the above reduction as follows:

$$V_R = [(h - x)/2L_s]\min(N; 0.55A_c f_c) + [1 - 0.05min(5; \mu_\Delta{}^{pl})] \cdot \{0.16max(0.5; 100\rho_{tot})[1 - 0.16min(5; L_s/h)]\sqrt{f_c}A_c + V_w\} \tag{2-1}$$

where h: is the depth of the cross-section (equal to the diameter *D* for circular sections); x: is the compressive zone depth; N: is the compressive axial force (positive, taken as being zero for tension); L_s: M/V ratio moment/shear at the end section; A_c: is the cross-section area, taken as being equal to $b_w d$ for a cross-section with a rectangular web of width (thickness) b_w and structural depth d or to $\pi D_c{}^2/4$ (where D_c is the diameter of the concrete core to the inside of the hoops) for circular sections; f_c: is the concrete compressive strength, and ρ_{tot}: is the total longitudinal reinforcement ratio.

For a typical reinforced concrete column (mean concrete strength of 30 MPa) with a 1.5 meter shear span (i.e., a clear height of 3.0m) and a 350 mm circular section (clear concrete cover 20mm) with 14Φ12 longitudinal reinforcement (yielding a strength of 500MPa) and Φ10/10 spiral reinforcement (yielding a strength of 500MPa) and axial load ratio of 20%, the axial load and concrete contribution to shear strength calculated based on the above equation (Eq. 2-1) lead to the following results: 49 kN axial load contribution which is the first term of the above equation (Eq. 2-1) and the concrete contribution is 34 kN. The reduction factor for a displacement ductility of 3 is 0.9. Therefore, the reduced concrete contribution is 31 KN.

For the same column under the same axial load and with the same material properties as above but comprised of a square section (457 mm) with 8Φ20 longitudinal reinforcement and Φ10/20 transverse reinforcement, the axial load contribution and the concrete contribution to shear strength are 137 kN and 98 kN respectively. The concrete contribution for displacement ductility equal to 3 will be reduced to the value of 88 kN.

In Eq. 2-1 term V_w is the contribution of transverse reinforcement to shear resistance, taken as equal to:

a) for cross-sections with a rectangular web of width b_w:

$$V_w = \rho_w b_w z f_{yw} \tag{2-2a}$$

where ρ_w is the transverse reinforcement ratio (Fig. 2-5); z is height of the equivalent truss, set equal to the internal lever arm, i.e., *d-d'* in beams and columns (Fig 2-5); and f_{yw} is the yield stress of the transverse reinforcement; and *S* the stirrup spacing.

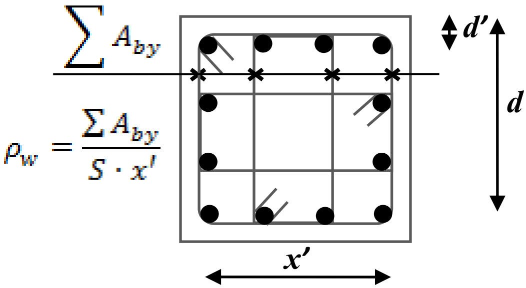

Figure 2-5: Transverse Reinforcement Ratio (*S*: spacing of the stirrups)

With regard to the example of the typical, square-sectioned column as described above based on Eq. 2-2a, the steel contribution in shear strength is 175 kN and the total shear strength of Eq. 2-1 is 410 kN. If the reduction factor is applied, the shear strength becomes equal to 383 kN. The variation of shear strength with spacing for this example under consideration leads to the following graph (Fig. 2-6).

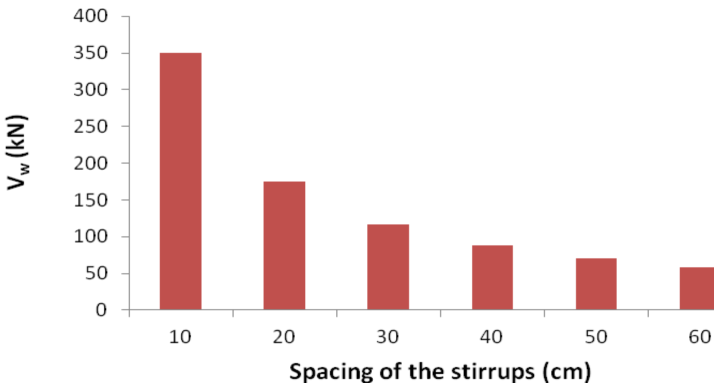

Figure 2-6: Effect of stirrup spacing to transverse steel contribution of a rectangular section in shear strength.

It is evident that for spacing greater than the effective depth of the section—which for the 45° degree truss analogy means that the shear crack doesn't intersect any stirrup—Eq. 2-2a simply leads to a lower value of steel contribution to shear strength. This is actually inconsistent – the value ought to be zero in this case; Pantazopoulou and Syntzirma (2010) have suggested that the term be substituted by,

$$V_w = \sum_{n_i} A_{swi} \cdot f_{si} \, ; n_i = [d/s] \; (greatest\ integer\ function) \tag{2-2b}$$

For circular cross-sections (c: is the concrete cover):

$$V_w = \frac{\pi}{2} \frac{A_{sw}}{s} f_{yw} (D - 2c) \tag{2-3}$$

Regarding the example of the typical column with the circular section as described above based on Eq. 2-3, the steel contribution in shear strength is 382 kN and the total shear strength of Eq. 2-1 is 465 kN. If the reduction factor is applied the shear strength becomes equal to 424 kN. The variation of shear strength with spacing for the example under consideration leads to the following graph (Fig. 2-7).

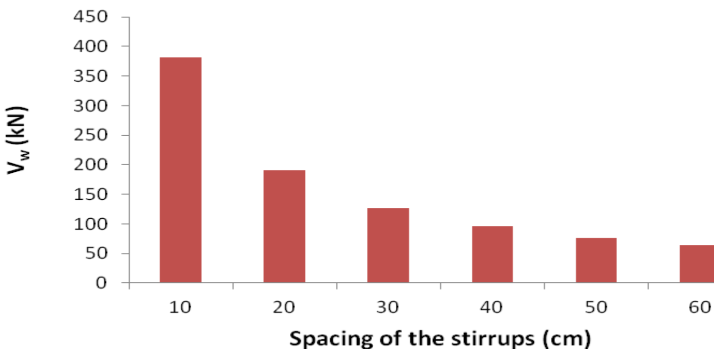

Figure 2-7: Effect of stirrup spacing on transverse steel contribution of a circular section in shear strength.

Based on Fig. 2-7, the steel contribution component should be based on the requirement that at least one stirrup layer must be intersected by the diagonal cracking plane; otherwise the steel contribution term ought to be taken as equal to zero.

In concrete columns with shear span ratio of L_s/h, less or equal to 2, the shear strength, V_R may not be taken as greater than the value corresponding to failure by web crushing along the diagonal of the column after flexural yielding, $V_{R,max}$, which under cyclic loading may be calculated from the expression:

$$V_{R,max} = (4/7)[1 - 0.02min(5; {\mu_\Delta}^{pl})][1 + 1.35(N/A_c f_c)][1 + 0.45(100\,\rho_{tot})]\sqrt{min(40; f_c)}b_w z \cdot sin2\delta \tag{2-4}$$

where δ is the angle between the cracking plane and the axis of the column ($\tan\delta = h/2L_s$). By implementing this equation to the example of the cases described above but with a change on the shear span so that the column be compliant to the shear span ratio limit of Eq. 2-4, the following results are obtained (L_s=700mm). It can be seen that for the circular column case shear strength is limited by web crushing along the diagonal.

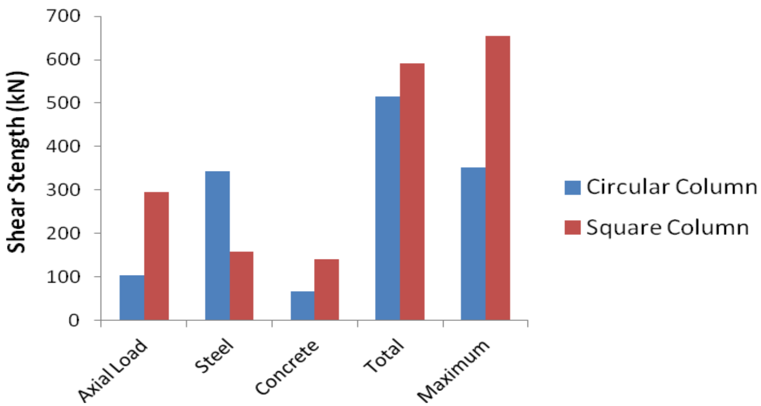

Figure 2-8: Shear Strength and its contributions for a typical reinforced concrete column.

ASCE/SEI 41 is the latest in a series of documents developed after the FEMA initiatives in the 1990s and 2000s towards the development of a consistent assessment framework for existing structures. The FEMA/ATC documents form the first integrated reference for performance-based engineering, whereby deformation and force demands for different seismic hazards are compared against the capacities at various performance limits (i.e. states of damage). At the outset of this momentous project by FEMA, available data on the performance of existing components were rather limited and therefore reliability concepts were not applied evenly towards the establishment of performance criteria.

The issue of dependably estimating the shear strength of a RC element appears to be rather complicated as it presumes the full understanding of the several interacting behavior mechanisms under reversed cyclic loading, whereas it is strongly affected by the imposed loading history, the dimensions of the element (e.g. the aspect ratio), the concrete strength, the longitudinal reinforcement ratio but mostly the ratio and the detailing of the transverse reinforcement. So far it has not been possible to theoretically describe the strength of the shear mechanism from first principles of mechanics without the use of calibrated empirical constants. Therefore the shear strength estimates obtained from calibrated design expressions necessarily rely on the pool of experimental data used for correlation of the empirical expressions, as well as on the preconceived notions of the individual researchers as to the role each variable has in the mechanics of shear.

The following expression for estimation of the shear strength of reinforced concrete columns is proposed by the Code for seismic rehabilitation of existing buildings of the American Society of Civil Engineers ASCE/SEI 41 (2007):

$$V_R = V_c + V_w = k(\mu_\Delta)\left[\left(0.5\sqrt{f_c}/(L_s/d)\right)\sqrt{1 + N/\left(0.5A_g\sqrt{f_c}\right)}\right]0.8A_g + k(\mu_\Delta)\left[A_{sw} f_{yw} d/S\right] \tag{2-5}$$

where $\boldsymbol{V_c}$ is the concrete contribution in shear resistance; $\boldsymbol{V_w}$ is the contribution of transverse reinforcement; d is the effective depth; $\boldsymbol{L_s}$ is shear span of the column; N is the axial force (compression positive, taken zero for tension); $\boldsymbol{A_g}$ is the gross cross-sectional area of the column; $\boldsymbol{A_{sw}}$ is the cross-sectional area of one layer of stirrup reinforcement parallel to the shear action; and $\boldsymbol{S}$ is the centerline spacing of stirrups. If *S* is equal to or greater than half of the effective depth of the column then the contribution of steel reinforcement V_w in shear strength is reduced to 50% of its estimated value from the above equation. If *S* is equal to or greater than the effective depth of the column then zero shear strength contribution from steel reinforcement V_w is considered; $\boldsymbol{f_c}$ is the concrete compressive strength; $\boldsymbol{k(\mu_\Delta)}$ is the shear strength reduction factor that depends on ductility. If ductility is less than or equal to 2 then the factor is set to equal to 1 (i.e. no strength reduction). If the ductility is greater than 6, then the reduction factor is equal to 0.6. For ductility between 2 and 6 the reduction factor is linearly interpolated between the proposed values.

The V_c estimate given by Eq. 2-5 for the example of the rectangular column presented in this Section is: $V_{c,ASCE}$ = 233 kN, while EN 1998-3 (2005) resulted in $V_{c,EC8\text{-}3}$= 88 kN which, when combined with the axial load component (137 kN) leads to a total of 225 kN, which is comparable to the result of Eq.2-5. For the case of the circular column results to $V_{c,\mathrm{ASCE}}$ = 81 kN whereas $V_{c,EC8\text{-}3}$=80 kN (49 kN axial load contribution+31 kN concrete contribution) – values calibrated well with each other.

The effect of the stirrups' spacing to the steel contribution to shear strength is depicted in the following figures for ASCE/SEI-41 (2007) and it is compared with the EN 1998-3 (2005) (here abbreviation EC8-III is used) results.

Despite the convergence of the calibrated expressions, the preceding comparisons highlight some of the uncertainties underlying the shear problem. For one, the concrete contribution term is taken—in both code documents—to be independent of the amount of transverse reinforcement, an omission that goes to the root of the truss-analogy model as originally introduced by Ritter and Moersch: there the concrete contribution component was thought to be a minor correction to the main component that was owing to transverse reinforcement (the truss posts) so as to improve correlation with the tests – it was never meant to be a component of commensurate importance and magnitude to that of transverse reinforcement. Another source of uncertainty lies in the treatment of the axial load: in the EN 1998-3 (2005) approach, the axial load contribution is dealt with as a separate term, whereas in the ASCE/SEI 41 (2007) approach it is treated as an offset to the tensile strength of concrete in the member web. This difference causes a departure in the V_c values near the upper limit in the axial load ratio (v=N/A_gf_c) as depicted in Fig. 2-11.