

ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE
ENGINEERING AND TECHNOLOGY

**EVALUATION OF PLASTIC HINGE LENGTH ESTIMATIONS
AND STRAIN LIMITS OF
REINFORCED CONCRETE SHEAR WALLS**



M.Sc. THESIS

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Department of Civil Engineering
Structural Engineering Programme

JUNE 2018

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

**BETONARME PERDE DUVARLAR İÇİN
PLASTİK MAFSAL BOYU YAKLAŞIMLARININ VE
BİRİM ŞEKİL DEĞİŞTİRME LİMİTLERİNİN DEĞERLENDİRİLMESİ**

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HAZİRAN 2018

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Date of Submission : 04 May 2018
Date of Defense : 04 June 2018





To my family,



ACKNOWLEDGEMENTS

I would like to express my gratitude to my thesis supervisor Prof. Dr. Unal Aldemir for the support he provided me during my thesis with his patience and knowledge.

If it had not been for Prof. Dr. Alper Ilki, this thesis would have never been a fulfilment, whose wisdom and knowledge expanded my horizons, whose patience and considered attitude make it easier at the most challenging times, whose valuable and constructive suggestions enlighten my path throughout the planning and development of this thesis. His willingness to give his time so generously has been very much appreciated. I would like to offer my special thanks to him.

Finally, I wish to give my deepest appreciation to my wonderful wife who always make me feel like the luckiest guy in the world. Without her endless support and patience, this study would have never been successful.

May 2018

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ABBREVIATIONS

ACI	: American Concrete Institute
ASCE	: American Society of Civil Engineers
NLFEA	: Non-linear Finite Element Analysis
RC	: Reinforced Concrete Elements
PHA	: Plastic Hinge Analysis
PHL	: Plastic Hinge Length





SYMBOLS

A_g	: Gross area of concrete section
BR	: Rupture of Previously Buckled Bars
CB	: Compressive Crushing of Concrete
c	: Neutral axis depth
c_c	: Clear cover to outside edge of reinforcement,
d_{bl}	: Nominal diameter of boundary longitudinal reinforcing bar
E_c	: Modulus of elasticity of concrete
E_s	: Modulus of elasticity of reinforcement
f'_c	: Specified compressive strength of concrete
f'_{cc}	: Specified compressive strength of confined concrete
f_y	: Specified yield strength of reinforcement
f_{yh}	: Specified yield strength of transverse reinforcement
f_{yw}	: Specified yield strength of boundary transverse reinforcement
h_w	: Effective height of wall
l_{bl}	: Length of confined boundary region measured horizontally
l_p	: Plastic hinge length
L_V	: Shear span
l_w	: Length of wall in direction of shear force
M	: Overturning moment at base
M_N	: Nominal flexural strength at section
M'_y	: Yield moment
P	: Axial load acting on cross section
RO	: Pure Tensile rupture of Reinforcing Bars
s	: Center-to-center spacing of transverse reinforcement
V	: Shear force demand on section
V_N	: Nominal shear strength
ϵ_c	: Extreme fiber compression strain
ϵ_{cu}	: Extreme fiber ultimate compression strain
ϵ_s	: Extreme fiber tension strain

ϵ_{su}	: Extreme fiber ultimate tension strain
ϵ_y	: Yield strain of reinforcement
ϕ_y	: Yield curvature
ϕ'_y	: First Yield curvature
ϕ_u	: Ultimate curvature
ϕ_p	: Inelastic curvature
Δ_f	: Lateral flexural displacement
Δ_y	: Lateral displacement at yielding of flexural reinforcement
Δ_p	: Lateral displacement after yielding of flexural reinforcement
Δ_s	: Lateral shear displacement
Δ_u	: Total lateral deformation
ρ_{sh}	: Ratio of distributed web transverse reinforcement to gross concrete area perpendicular to that reinforcement
ρ_b	: Boundary element reinforcing ratio
ρ_s	: Boundary volumetric transverse reinforcing ratio

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EVALUATION OF PLASTIC HINGE LENGTH ESTIMATIONS AND STRAIN LIMITS OF REINFORCED CONCRETE SHEAR WALLS

SUMMARY

Lateral displacement of a reinforced concrete shear wall is basically comprised of two primary parts which are shear and flexure. Flexural part constitutes the larger portion of total displacements that includes contributions from both elastic and plastic branch of the response whereas shear part is mostly overlooked for slender walls for which the shear-flexure interaction is not expected to have a profound influence in overall behaviour.

To assess the global scale deformation capacity of a reinforced concrete shear wall, there are several simulation/modelling techniques proposed in the literature. Plastic hinge analysis (PHA), is one of the simplest and widely utilized numerical methods to predict displacement capacity or to determine local damage state for a specific drift demand imposed by ground motions.

The efficiency of the assessment via plastic hinge analysis method immensely depends on adopting the appropriate estimations of the plastic hinge length and the strain limits. Typical building codes and several researchers recommend to take the plastic hinge length at the wall base as the half of the wall length. However, for most of the cases, this simplicity may yield unreliable results since the plastic hinge length is not only impacted by wall length, but also influenced by several more parameters which will be discussed further. Another concern is given to proposed strain limits of which applicability to shear walls is a huge question mark on the ground that most of them are calibrated for pure axial compression or tension effects.

To address these issues, a dataset of 49 shear wall test is assembled and studied for the validation of plastic hinge analysis method. Subsequent to plastic hinge analysis method, a moment-curvature analysis was performed for all specimens.

The major outcomes of the study are as follows;

- Adopting the right failure criterion based on anticipated failure mode as means of predicting the displacement capacity is of crucial. A new expression is proposed to predict the failure mode of shear wall which utilizes section analysis results.
- The performance evaluation procedure based on compression side strain limits for shear walls is related to large uncertainties and compressive strain limits are not as dependable as those for reinforcing bars.
- The use of the equivalent plastic hinge length to half the wall length may jeopardize the reliability of the PHA method. The estimation of Eurocode 8, in general, well agrees with the experimental results.



BETONARME PERDE DUVARLAR İÇİN PLASTİK MAFSAL BOYU YAKLAŞIMLARININ VE BİRİM ŞEKİL DEĞİŞTİRME LİMİTLERİNİN DEĞERLENDİRİLMESİ

ÖZET

Deprem etkisi altında betonarme binaların/elemanların kuvvet esaslı dayanıma göre tasarlanması yaklaşımı son çeyrek asırda yerini şekil değiştirmeye göre tasarım ve değerlendirme yaklaşımına bırakmaktadır. Dayanıma göre tasarımın en büyük dezavantajı olarak, tasarımda göz önüne alınan deprem etkisi, bina servis ömrü sırasında tecrübe edildiğinde binanın bu etki karşısında nasıl bir performans göstereceği hakkında tasarımcıya herhangi bir fikir vermemesi gösterilebilir. Şekil değiştirmeye göre değerlendirmede ise binanın hedeflenen performans seviyelerini ne ölçüde sağladığı/sağlamadığı kestirilip ona göre ön tasarımda iyileştirmeler yada mevcut yapılar söz konusuysa güçlendirme yöntemleri ile çözüm aranabilir.

Betonarme perde duvarlar yatay yükler karşısında önemli bir rijitlik, dayanım ve stabilite sağladığı için deprem kuşağında yeralan coğrafyalarda, bina tasarımında sıklıkla tercih edilmektedir. Betonarme diğer elemanlara nazaran da daha büyük bir rijitliğe sahip olduğu için de bina davranışına genel anlamda hükmeden perde duvarlar olmaktadır. Bu sebeple perde duvarların yatay yerdeğiştirme kapasitelerinin ya da belirli bir yerdeğiştirme etkisi altında lokal birim şekil değiştirme istemlerinin kestirilmesi bina performansının belirlenmesi açısından önem arz etmektedir. Bu amaçla doğrusal olmayan davranış modelleri geliştirilip perdelerin analitik olarak modellenmesi hedeflenmiştir. Bu davranış modellerinden en yaygın kullanılanlardan birisi yığılı plastik davranış modelidir. Bu modelin en önemli faraziyesi, akma kapasitelerine erişen bölgelerde plastik şekil değiştirmelerin düzgün yayılı bir şekilde belirli bir uzunlukta toplandığıdır. Bu uzunluk plastik mafsalsal uzunluğu olarak isimlendirilir.

Yatay yerdeğiştirme temel olarak, eğilme ve kesme olmak üzere iki bileşenden oluşmaktadır. Eğilme bileşeni, narin perdelerde genel olarak oransal anlamda daha büyük bir paya sahipken, kesme bileşeni ise daha küçük bir paya sahiptir. Bu sebeple kesme bileşeninin narin perdelerde çoğu zaman ihmal edilmesi gibi bir eğilim vardır. Kesme bileşenin tepe deplasmanına katkısının hesaplanması için literatürde mevcut birkaç yaklaşım bulunmaktadır. Eğilme sebebiyle oluşan yatay deplasman da kendi içinde elastik ve inelastik kısım olarak ikiye ayrılır. Elastik kısım, elastik teoriye göre, akma eğriliği ve perde boyu ile hesaplanırken, plastik olan kısımda ise yığılı plastik mafsalsal metodunda yapılan kabuller devreye girer. Plastik eğrilik plastik mafsalsal boyu ile çarpılıp plastik dönmeler elde edilir. Bu dönmeler ise perde gibi konsol çalışan elemanlarda konsol boyu ile çarpılıp yatay yerdeğiştirmelerin elastik ötesi bölümü elde edilmiş olur. Bu tanımdan da anlaşılacağı üzere perdenin yatay ötelenme kapasitesini asıl olarak üç faktör belirlemektedir. Bunlar, birim şekil değiştirme, tarafsız eksen derinliği ve plastik mafsalsal uzunluğudur. Bilindiği üzere birim şekil değiştirmenin tarafsız eksen derinliğine oranı kesit eğriliğinin ifadesidir.

Tarafsız eksen derinliğinin tespiti, düzlem kesit şekil değiştirmeden sonra da düzlem kalır faraziyesiyle, kesit analizinden yararlanılarak nispeten daha kolay şekilde yapılabilir. Şunu unutmamak gerekir ki, bu kabul olmadan tarafsız eksen tanımı ve kavramı bambaşka bir hal alabilmektedir. Bu çalışmada düzlem kesit düzlem kalır hipotezi baz alınarak kesit analizi yapılmıştır.

Plastik mafsalsal modelinde plastik mafsalsal uzunluğu, plastisitenin yayıldığı bölgeyi bu bölgenin eşdeğeri olarak temsil eder. Elastik ötesi davranışın perde boyunca yayıldığı bu bölge 3 fiziksel olgunun bileşimi olarak ortaya çıkar. Bunlardan birincisi, “moment gradient” olarak ifade edilen moment diagramındaki eğimin plastik mafsalsal uzunluğuna etkisi. İkincisi ise, “tension-shift” etkisi. Bu etki, eğilme-kesme etkileşimi sonucu ortaya çıkan eğilme-kesme çatlakları sebebiyle en büyük momentin, mesnet bölgesinden, ortaya (kirişte) ya da perde durumunda yukarıya doğru ötelenmesi sebebiyle plastikleşen bölgenin büyümesi olarak tanımlanabilir. Üçüncü ve son olarak ise “strain-penetration” etkisi, akma durumu için donatı sıyrılması dönmesini ifade eden bu etki de plastik mafsalsal boyuna dahil edilerek plastik mafsalsal boyu belirlenmektedir.

Bu parametrelerin dışında, eksenel yük etkisi, donatıda pekleşme etkisi, kesme kuvvetinin etkisi gibi birçok faktör araştırmacılar ve araştırma grupları tarafından çalışılsa da plastik mafsalsal boyu yaklaşımlarında bu etkilerin hesaba dahil edilip edilmemesi hala netlik kazanmamıştır.

Plastik mafsalsal modelinde kullanılacak birim şekil değiştirme limitlerin tayini ise bir diğer önemli husustur. Literatürde, gerek basınç tarafı, gerekse çekme tarafı için önerilen birim şekil değiştirme limitleri olsa da, bu limitlerin betonarme perde duvarlar için plastik mafsalsal modelinde güvenli bir şekilde kullanılıp kullanılamayacağı sorusu hala cevap beklemektedir. Plastik mafsalsal modelinde, sargılı beton için tanımlanmış birim şekil değiştirme limiti, salt eksenel yük etkileri altında sargı donatısının kopma anına kadar tüketilen enerjinin denklighinden türetilmiş olup premature donatı burkulması durumlarında makul sonuçlar vermediği bilinmektedir. Çevrimsel yükler altında, hem eksenel hem de moment etkisi altında olan ve her ne kadar önlenmeye çalışılsa da boyuna donatılarda burkulmanın gözlemlendiği perde duvarlarda, bu limitler kullanılarak konservatif olmayan perde yatay kapasitesi tahminleri elde edilmiştir.

Birim şekil değiştirme limitinin güvenilirliğinin yanısıra eğilme göçmesinin hangi tarafta olacağının kestirilmesi de plastik mafsalsal modelinin doğruluğu açısından büyük önem taşımaktadır. Şöyle ki, çevrimsel yük etkisi sebebiyle basınç tarafında daha önce burkulmuş donatının çekme tarafında kopması sebebiyle göçme öncesi durumunu tecrübe etmiş bir betonarme perde deney numunesi için, basınç tarafı birim şekil değiştirme limitleri doğru sonuçlar vermeyecektir. Ya da basınç tarafında beton ezilmesi durumunu yaşamış bir perdenin kapasitesi, çekme tarafı limitleri ile doğru ve anlamlı bir şekilde belirlenemeyecektir.

Tezin organizasyonu anlamında;

1. Bölüm’de çalışmanın önemi, kapsamı ve amacı ifade edilmeye çalışılmıştır.
2. Bölüm’de konu ile ilgili terminolojinin literatür karşılıkları incelenmiştir. Akma eğriliği, plastik eğrilik, birim şekil değiştirme limitleri gibi tanımların yanısıra farklı

araştırmacılar tarafından önerilen 11 farklı plastik mafsalsal boy yaklaşımı ve plastik mafsalsal metodunun detayları verilmiştir.

3. Bölüm’de bu çalışma kapsamında incelenen perde deneylerinin bir kısmı detaylı bir şekilde incelenmiştir. Kesit boyutları, donatı düzeni, test günü malzeme sonuçları, test düzeneği gibi deney öncesi datalarla beraber, deney sonuçlarına ait histerez döngü grafikleri, hasar ilerleme raporları ve nihai hasar durumuna ait bilgiler ilave şekiller ve çizelgelerle beraber sunulmuştur.

4. Bölüm’de eğilme davranışının hakim olduğu 49 perde deneyi baz alınarak, plastik mafsalsal modeli kullanılarak birim şekildeğiştirme limitlerinin ve bu limitler ile beraber kullanılan plastik mafsalsal boyu yaklaşımlarının deney sonuçlarına karşı geçerliliği sınanmıştır. Bu bağlamda perde duvarların modellenmesi ve değerlendirilmesi için gerekli dataların (malzeme, kesit, geometri, yükleme ve şekil değiştirme özellikleri) toplanması suretiyle perde veri tabanı oluşturulmuştur. “OpenSees” platformu kullanılarak perde numunelerinin kesit analizleri yapılmış moment-eğrilik ilişkileri elde edilmiştir. Literatürde geçerliliği olan birim şekil değiştirme limitleri ve plastik mafsalsal uzunluklarının farklı kombinasyonları kullanılarak elde edilen yatay yerdeğiştirmeye değerleri deney sonuçları ile karşılaştırılmıştır.

5. Bölüm’de ise aşağıda özetlenen sonuçlar verilmiştir;

- Kırılma tipini kesit analizinden tahmin etmek üzere bir yaklaşım geliştirmiştir. Bu sayede kırılma moduna göre uygun birim şekildeğiştirme ve plastik mafsalsal uzunluğu seçilip daha yakınsak yatay deplasman kapasitesi tahmini yapılması hedeflenmektedir.
- Birim şekildeğiştirme limitlerinin güvenilirliği sınanmıştır. Sargılı beton için için verilen birim şekil değiştirme limitlerinin perdelerde, genel anlamda donatı için verilen birim şekil değiştirme limitleri kadar güvenilir olmadığı görülmüştür.
- Plastik mafsalsal boy yaklaşımlarının geçerliliği değerlendirilmiştir.. Klasik yaklaşım olan plastik mafsalsal boyunu perde boyunun yarısı olarak kabul etmenin plastik mafsalsal analiz metodunun güvenilirliğini tehlikeye attığı sonucuna varılmıştır.



1. OVERVIEW

For the last two or three decades, design methodologies & provisions for reinforced concrete members have shown a trend which prioritize displacement based approaches since designing on the basis which focus on meeting the required strength demands for minor/frequent seismic actions would not provide adequate safety in the event of strong/rare earthquake motions. Given the fact that, satisfying the demand of major earthquakes with strength capacities would not be feasible economically, achieving a significant ductility capacity for RC structures/members has become the major concern of practising engineers and researchers.

Shear walls efficiently provide high lateral stability, rigidity and strength for a RC building that is prone to experience relatively strong ground motions. As a means of assessment and anticipation of damages likely to be given by seismic effects to shear walls, simulation methods have been developed over the years by making use of numerical modeling techniques. The general aim of all the modeling approaches is to capture a global scale force-displacement relationship as well as identifying local scale demands such as curvatures and strains. Plastic hinge modeling, one of those approaches, is a simple analytical tool which bridges local scale deformations with global scale ones. The plastic deformation with plastic hinge modeling can basically be expressed as follows;

$$\Delta_p / (h_w - l_p / 2) = (\frac{\epsilon_{cu}}{c} - \phi_y) * l_p \quad (1.1)$$

where Δ_p stands for plastic part of the lateral displacement, h_w is the shear span or the wall height, l_p denotes for the plastic hinge length and ϵ_{cu} is ultimate compressive concrete strain limit.

1.1 Research Significance

Plastic hinge modeling approach simply rules that flexural part of the lateral displacement is directly related to the three terms ϵ_{cu} , c and l_p . The term c , of the

three, is the most apparent and the easiest one to determine, provided that section compression depth c is obtained via section analysis on the basis of Bernoulli Hypothesis of plane section remains plane. However it is noteworthy to state that regardless of this assumption, defining c would also get more elaborate which is out of scope of this research.

Plastic hinge length l_p characterizes the length over which spread of plasticity extends, which principally includes contributions from following three effects; moment gradient effect, tension shift effect and strain penetration effect. Plastic hinge estimations basically are product of varying combinations of these three contributions. Although many other parameters, which may have an influence in propagation of plastic zone, i.e axial force, strain hardening, have been studied by researchers, it yet remains unclear whether to include them in estimations of plastic hinge length or not.

Defining a strain limit to employ in plastic hinge modeling approach as to predict displacement capacity of a wall is the most challenging of all. Although many local damage limits are proposed in the literature which indicates failures both in compression and tension side, their reliability is immensely questionable due to many uncertainties which will be further dissussed.

One of the key aspects to complete the assessment of displacement capacity is that the shear displacements need to be incorporated in plastic hinge modeling since they can constitute a significant portion of the total displacements.

Moreover it should be noted that, the above mentioned procedure can only represent the force-displacement relation unless any other failure modes rather than flexure, i.e diagonal strut failure, diagonal tension failure, shear sliding or out of plane instability are anticipated. To distinguish flexural failure mode from the others, for instance, the hardest one to differentiate, diagonal compression failure (strut), one needs to apply one of the various approaches presented by different research groups (Hines and Seible, 2004 and Oesterle et al., 2004) especially when flexure-shear interaction is expected to dominate the behavior.

In light of above explained issues, the accuracy of the assessment of force-displacement relations through plastic hinge modeling highly depends on the harmony and appropriateness of the strain limits and plastic hinge length estimations of which will be employed in conjunction with each other.

1.2 Objective and Scope of the Study

With the objectives listed in the following paragraph, a database of 49 laboratory shear wall specimens of which failure mode is mainly either flexural compression i.e compression zone concrete crushing and longitudinal reinforcing bar buckling or flexural tension i.e rupture of boundary bars were assembled and studied. Database include both fully~ and semi fully code compliant specimens as to also investigate the effect of variables which violates design codes. However, other failure modes such as shear compression/tension, shear sliding, lap splice and out of plane buckling failure have been precluded in order to avoid complication.

Objectives;

- A new procedure, to predict failure modes prior to applying plastic hinge modeling will be developed based on the section analysis results of a set of 49 experimental wall specimens, which will serve for the best interest of assessment purposes (i.e predicting the drift capacity of walls of which behavior governed by flexure) since adopting the pertinent failure criterion is highly essential.
- The reliability of existing strain limits used in performance assessment in terms of determining drift capacity will be investigated through plastic hinge modeling by validating against the shear wall database.
- The other key aspect of plastic hinge modeling is to establish a plastic hinge length. Validation of current state of the art plastic hinge length proposals will be made against shear wall test results.



2. LITERATURE REVIEW ON BASIC TERMINOLOGY

How to obtain and evaluate local responses of shear walls as a meaning of seismic assessment procedures?

In terms of analytical approaches, specifically plastic hinge modeling for the purpose that is either calculating local demands or global capacities; Local response of a structural member can be determined by conducting section analysis on the basis of plane section assumption by taking into account axial forces, cross-section properties, reinforcement layouts and material properties etc. The output of a section analysis which is hereby called as local response is, curvature and strain values interrelated to specific moment values which, in case of cantilever walls, can be easily transformed to shear forces by given shear span values.

For a given specific global response (drift ratio), applying the plastic hinge modeling and assuming basic $l_p = 0.5h$, one can determine the plastic curvature demand easily or vice versa, for a given local deformation limit, for say, ultimate strain, following the above mention method the other way around, global deformation capacity can be specified.

In most of the codes, deformation limits of structural members are given in terms of local responses (rotation, curvature or strain) as well as global responses (drift ratios)

On the other hand, state-of-the-art macro or micro modeling techniques provide many response parameters both global and local scale together. However in micro modeling i.e finite element analysis, to make a proper comparison with code-type strain limits which rely on plane section analysis, total strain excursions needs to be converted to a curvature value and then strains corresponding to that curvature can be calculated from section-analysis.

Utilizing section analysis is also required, when it comes to evaluating measured local strains of an experiment, the obtained local results via gauges shall not be used directly as damage indicator or else unless experimentally measured strains or curvatures are

processed through a section analysis as to be compatible with conventional analyses procedures and deformation measurements specified in codes and standards.

2.1 Yield point, first yield, global yield, bilinearization

Yield point represents the end of the elastic branch of the behaviour which may be called as the global onset of the inelastic part.

First yield is the instant where the extreme fiber in the tensile zone reaches $\varepsilon_s = f_y/E_s$, most of the time, estimating the elastic contribution of flexural displacement with the first yield is conservative.

Yielding of a section can be determined by various approaches reported in the literature. One of the most adopted approach is that the outer bar strain's reaching $\varepsilon_s = 0.003$ indicates yielding or some propose to take the point when the middle fiber reaches $\varepsilon_s = f_y/E_s$ as a global yield point.

2.2 Ways to determine Yield Curvature

Determining the yield curvature on a bilinearized load deformation curve is of importance for not only the reason that the equivalent yield curvature is prerequisite for calculating the elastic contribution of flexural deformation but also the serviceability limit state in many codes is mostly defined according to that. Several methods have been proposed in the literature to define equivalent curvature.

Paulay and Priestley (1992) defined an equivalent yield curvature, which is obtained by scaling the yield curvature corresponding to the first yield of the tensile reinforcement at the extreme fiber ($\varepsilon_s = f_y/E_s$) for bilinearization of moment-curvature curve. The coefficient is defined as the ratio of the nominal moment (M_N) to the yield moment at first yield M'_y . Here M_N holds for the nominal moment (M_N) that is defined as the moment where the extreme fiber strain in compression reaches 0.004 or extreme tension strain reaches 0.015, whichever occurs first. The bilinear curve is obtained by drawing the first line from the origin to the point on the curve where the reinforcement yields for the first time extending up to the nominal yield moment. The second line connects the first line at the top to the ultimate point on the moment-curvature curve.

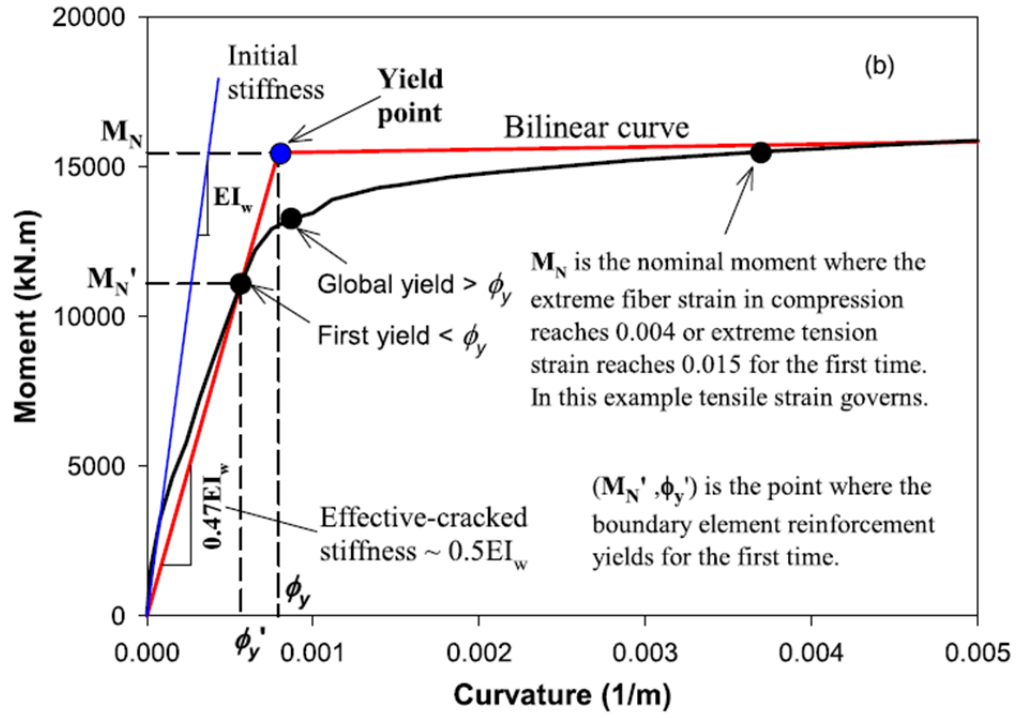


Figure 2.1 : Bilinearization of yield curvature. (Kazaz I. 2013).

$$\phi_y = \phi_y' \frac{M_N}{M_y'} \quad (2.1)$$

Based on above procedure, Priestley et al. (2007) suggested the following empirical curvature expression (equation 2.2) to calculate the curvature for serviceability limit state

$$\phi_y = 2 \frac{\varepsilon_y}{l_w} \quad (2.2)$$

One other empirical proposal for the estimation of elastic curvature is made by Massone et al. (2016) which aims to take into account the axial force and boundary zone vertical reinforcement effect by factoring the $\frac{\varepsilon_y}{l_w}$ term by a coefficient “K”. The formula is given in (equation 2.3) ;

$$\phi_y = K \frac{\varepsilon_y}{l_w} \quad (2.3)$$

Where “K” basically stands for the influence of section geometry, the distribution of cracking over the height, the level of axial load, the boundary reinforcement on yield curvature. The factor K can be parameterized in terms of these two variables (equation 2.4).

$$K = 1.25 + 1.69 \frac{P}{A_g f'_c} + 0.65 \rho_b \quad (2.4)$$

Massone et al. (2016) also states that a conservative $K = 1.40$ can be considered.

Kazaz et al. (2012) chose a different approach to obtain yield curvature. The proposal is the outcome of regression analysis carried out which is based on the numerical models of the set of walls with varying parameters (equation 2.5).

$$\phi_y = 0.00534 \frac{\rho_b^{0.7}}{l_w} \quad (2.5)$$

Table 2.1: Yield curvature estimations.

$\phi_y = \phi'_y \frac{M_N}{M'_y}$	Paulay and Priestley (1992)
$\phi_y = 2 \frac{\varepsilon_y}{l_w}$	Priestley et al. (2007)
$\phi_y = K \frac{\varepsilon_y}{l_w} ; K = 1.25 + 1.69 \frac{P}{A_g f'_c} + 0.65 \rho_b$	Massone et al. (2016)
$\phi_y = 0.00534 \frac{\rho_b^{0.7}}{l_w}$	Kazaz et al. (2012)

2.3 An Ultimate Point Problem

Predicting the displacement capacity of a structure has been no doubt one of the main concerns of researchers for many years. Hence, the comparison of analytical and experimental studies is widely preferred by researchers as to verify their findings. In terms of global responses of shear walls, the point where the peak load drops by 15% or 20% on the load deformation curve or hysteresis loop is assumed to be the ultimate point and the corresponding deformation value is called as the ultimate displacement or the lateral displacement capacity of the wall.

For section level responses, different research groups have suggested to adopt various strain or curvature limits as the ultimate point, all of which indicates local damages such as buckling or rupture of reinforcing bar or crushing of concrete.

2.4 Ultimate compressive strain

Priestley et al. (2007) suggested a damage control compression strain which indicates the instant where the rupture of confining bar is likely to occur. It is based on equating the post-peak strain energy of concrete with the strain energy of confining reinforcement just before rupture takes place and also stands on the confined concrete model of Mander et al. (1988) which estimates the ultimate strain of unconfined concrete as $\varepsilon_c = 0.004$. The expression is as follows (equation 2.6);

$$\varepsilon_{cu} = 0.004 + 1.4 \frac{\rho_s f_{yw} \varepsilon_{su}}{f'_{cc}} \quad (2.6)$$

Where f'_{cc} is the compressive strength of confined concrete, ρ_s , f_{yw} , ε_{su} are the volumetric ratio, the yield strength and the maximum strain of confining reinforcement, respectively.

Biskinis and Fardis (2010) put to use a comprehensive experimental database and take up the obtained drifts where the peak load capacity drop by 20% to derive a strain limit. Thus this limit actually does not represent a local damage state but is directly linked to a global scale deformation particularly the reduced shear capacity. Furthermore, the impact of the confinement and the size of confined area is included in their formula;

$$\varepsilon_{cu} = 0.0035 + \left(\frac{1}{x_{c,con}} \right)^{1.5} + 0.4 \frac{\rho_s f_{yw} k_{con}}{f'_{cc}} \quad (2.7)$$

where $x_{c,con}$ is the depth of the confined part of the compression zone and k_{con} holds for confinement effectiveness factor according to Sheikh and Uzumeri (1982) which considers the spacing of the reinforcement in conjunction with the dimensions of the section.

Scott et al. (1982) derived an empirical equation for ultimate strain capacity of confined concrete members as the result of series of test of 25 square concrete confined columns. They found that the ultimate strain of concrete extreme fiber can be adopted as the strain value where the first hoop fracture takes place. the formula is;

$$\varepsilon_{cu} = 0.004 + 0.9 \rho_s \frac{f_{yw}}{300} \quad (2.8)$$

where ρ_v is the volumetric transverse reinforcement ratio and f_{yh} is the yield strength of transverse reinforcement. In these tests, the ultimate strain values varied between 0.0017 and 0.038.

2.5 Ultimate tensile strain

The failure of vertical rebars is generally assorted dependantly of buckling mechanism. Whether the rupture follows buckling or not is the main criteria for classifying failures of bars.

- The rupture mechanism in tension which does not follow any buckling. Generally seen in situations where unconfined cover has not spalled off yet.
- Having been buckled in compression, the rupture of bars occur in tension. This type of failure mostly is observed for the vertical bars which are prone to higher axial stresses at boundary zones.

Restrepo (1993) proposed that total strain excursions $\varepsilon_s - \varepsilon_c$ is better fitted as a failure criteria rather than employing ε_s solely. As a bar fracture strain limit Restrepo (2006) recommends following expression;

$$\varepsilon_s - \varepsilon_c = \frac{14 - \frac{4s}{3d_{bl}}}{100} \leq \frac{\varepsilon_{su}}{2} \text{ and } \varepsilon_c \leq -0.004 \quad (2.9)$$

Where s_h is the spacing of the stabilizing bars along the wall and d_{bl} is the diameter of the longitudinal reinforcement bars.

Restrepo (2006) propose using the next equation for the onset of buckling;

$$\varepsilon_s - \varepsilon_c = \frac{10 - \frac{s}{d_{bl}}}{100} \quad (2.10)$$

Priestley et al (2007) suggested a simple tensile strain limit claiming that computation of total excursions is rather hard when applying plastic hinge modeling which is;

$$\varepsilon_s = 0.6 \varepsilon_{su} \quad (2.11)$$

The reduction factor 0.6 for ε_{su} is employed to include not only the cyclic load reversals effect for which the reinforcing bar turns more susceptible to buckling but also the tension shift effect.

Besides Hines (2002), from his experiments on bridge piers, he postulated that the absolute sum of strain pairs, $|\varepsilon_s| + |\varepsilon_c|$, i.e the total excursions of strains, ranged between 0.035 and 0.05 lead to the onset of buckling.

$$0.035 \leq |\varepsilon_s| + |\varepsilon_c| \leq 0.05 \quad (2.12)$$

2.6 Ultimate curvatures

Kazaz et al. (2012) proposed an ultimate deformation limit in the form of dimensionless curvature, derived from a comprehensive analytical study investigating the influence of various parameters that is wall length, axial load ratio, shear span to wall length ratio and horizontal web reinforcement on shear walls deformation capacity by conducting Nonlinear Finite Element Analysis (NLFEA).

They found that wall length is the fundamental parameter affecting the ultimate curvature as in the case of yield curvature. Hence, for the sake of regression analysis, they preferred to modificate the term to a dimensionless form by multiplying it with wall length. The equation is;

$$\phi_u l_w = 0.8(C_L C_S) \varepsilon_{su} \left(1 - 2.4 \frac{P}{A_g f'_c}\right) \left(1 - 1.5 \frac{f_{yh}}{f'_c} \rho_{sh}\right) \left(M/V/l_w\right)^{0.29} \quad (2.13)$$

In the equation, C_L coefficient denotes for the degrading effect of the cyclic loading. Previous research have shown that the total deformation under cyclic loading might be 75% of the deformation under monotonic loading. Thus the authors recommend taking to C_L as 0.75 and 1.0 in the case of monotonic loading and cyclic loading, respectively.

The C_S term above, intends to account empirically for the cross-section shape which significantly affects the behaviour of shear walls. Past experimental studies have demonstrated that Barbell shaped shear walls have larger deformation capacity comparing to those which have rectangular section. Kazaz et al. (2012) propose to take the values of 1.0 and 1.25 for rectangular and barbell section walls, respectively.

Priestley et al. (2007) reported that the dimensionless curvature for damage control limit state $\phi_{dc} l_w$ can be adopted with constant average value of 0.072;

$$\phi_{dc} l_w = 0.072 \quad (2.14)$$

2.7 Lumped Plasticity Models

The lumped plasticity approach is simply based on the assumption that the post-elastic branch of the deformation can be characterized by a so called plastic-hinge which is located near the section where the moment demand is expected to be the highest and also the curvature does not vary in that plasticity region.

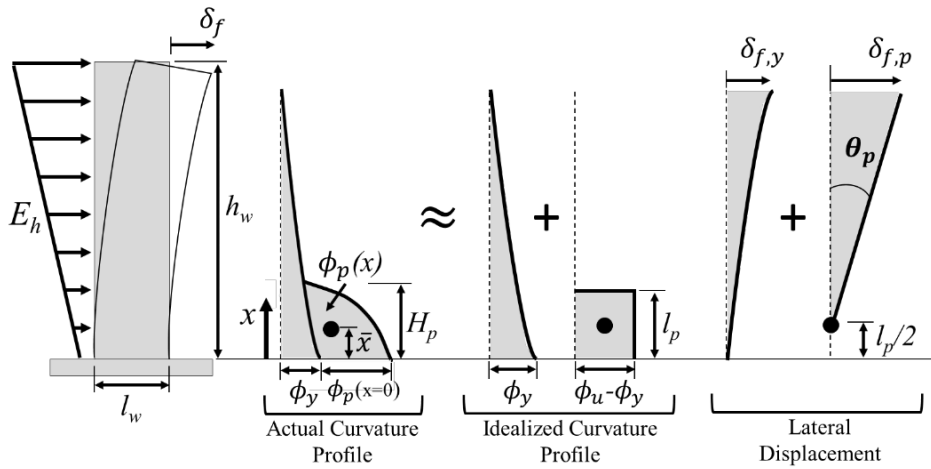


Figure 2.2 : Plastic Hinge Analysis theoretical curvature distribution due to flexural deformations and idealized curvature distribution with lateral displacements.

Although plastic hinge modeling approach reserves some shortcomings of representing the actual physical behavior of RC members, it is widely accepted amongst researchers and design engineers for the sake of its simplicity.

Prior to the application of the plastic hinge modeling approach to plot a force-displacement curve, it is essential to determine the yield and ultimate curvatures typically from section analysis and to pick a plastic hinge length from currently available plastic hinge-length proposals. One should note that, the closer prediction of the displacement is mostly dependant on the consistency of the combination of the appropriate strain limits in the form of corresponding curvatures and the suitable plastic hinge length. The simplest and widely used form of plastic deformation by Paulay and Priestley (1992), is as follows;

$$\Delta_p = (\phi_p * l_p) * (h_w - l_p/2) \quad (2.15)$$

Where ϕ_p denotes for the plastic curvature that can be written in this form $\phi_p = \phi_u - \phi_y$, h_w is the shear span of a cantilever wall and l_p is the plastic hinge length which is generally taken as equal to the half of the length of the wall. In the above equation, the rotation symbolizing the plastic deformation is computed by simply integrating the plastic curvature along the height of the plastic hinge and is considered to be at midheight of the plastic hinge. However Hines (2002) noted that a centre rotation at midheight of plastic hinge is a more suitable assumption for design purposes and a centre rotation at the base can be adopted for assessment purposes.

$$\Delta_p = (\phi_p * l_p) * (h_w) \quad (2.16)$$

The total flexural deformation is comprised of plastic deformations and elastic deformations. Paulay and Priestley (1992) propose an estimation for the elastic deformations i.e the yield displacements of a cantilever wall, which basically assumes a triangular linear distribution of curvature along the height of wall and can be written as:

$$\Delta_y = (\phi_y * h_w^2/3) \quad (2.17)$$

2.8 Plastic hinge length

In this section, the models developed by various researchers to determine the plastic hinge length will be presented. The need to establish a proper plastic hinge length stems from the idea that the assumption of elasto-plastic model under the Bernoulli-hypothesis of plane section remain plane with a simple plastic hinge length (l_p) as the half of the wall length (l_w) does not hold for the parameters which effect the spread of plasticity. Those phenomenas expilicltly explained by Priestley and Park (1987), which are moment gradient effect, tension shift and strain penetration effect.

The increase in capacity between the moment at first yield M'_y and the ultimate moment M_u combined with the moment gradient results in spreading of the plastic deformations over a certain length of the structural member. The length of the region over which plasticity spreads due to this mechanism is commonly expressed as a ratio of the effective height h_w while the ratio of M'_y/M_N , i.e. the hardening of the section, is often not explicitly taken into account (moment gradient).

The inclined flexural-shear cracks lead also to a spread of the plastic deformation since the steel strains above the section of the largest moment will be increased (tension shift).

Strain penetration along the reinforcement bars into the adjacent elastic member (e.g. the foundation) is the third mechanism that increases the plastic deformation capacity of the structural member. The strain along the anchorage length of the reinforcing bar in the foundation leads to a slip between the reinforcing bar and the foundation, which results in uplift at the structural wall or column base implying additional base rotation. This can be expressed as the product of a plastic curvature times an additional plastic hinge length increment.

Priestley and Park (1987) put forward an estimate of plastic hinge lenge which accounts for the contribution of the three abovementioned factors. Their equation in a general form can be written as follows;

$$l_p = A h_w + B l_w + C d_{bl} \quad (2.18)$$

Where d_{bl} is the boundary longitudinal bar diameter, l_w is the wall length, h_w is the effective height of the wall and A, B, C are the constants. "A" denotes for the

contribution of the moment gradient effect, “B” represents the tension shift effect while “C” remarks the impact of strain penetration. Most of the state of the art plastic hinge length proposals are derived from the general equation given above with the varying values of A, B, C constants all of which consider the each contribution to the spread of plasticity to different extents. Paulay and Priestley presented the following expression to estimate plastic hinge length Priestley, Seible and Calvi (1996 : 308-309):

$$l_p = 0.08 h_w + 0.022 f_y d_{bl} \geq 0.044 f_y d_{bl} \quad (2.19)$$

Where f_y is in MPa units. the authors also indicated that above equation gives plastic hinge lengths of approximately $0.5l_w$. In this formula, they intend to consider the contribution of both moment gradient effect and strain penetration effects but they excluded tension shift parameter.

Paulay and Priestley (1993) reported tests on ductile RC walls of rectangular cross-section which subjected to seismic loading to investigate the out of plane buckling phenomena. They derived the following equation for the plastic hinge length (Paulay and Priestley, 1993: 386-387) :

$$l_p = 0.2 l_w + 0.044 h_w \quad (2.20)$$

Where h_w is the effective height of the wall. The authors indicate that equation (2.20) yields conservatively the plastic hinge length.

Paulay and Priestley (1992) also noted that “the plastic hinge length, for slender walls, typically equals to 30% to 80% of the wall length l_w ”. Priestley and Kowalsky (2000) also suggest to take the larger of the above given two equations. Priestley et al. (2007) come up with a fresh and revised version of the general equation which accounts for the three of the contributions to some degree;

$$l_p = k h_w + 0.1 l_w + 0.022 f_y d_{bl} \quad (2.21)$$

Where k is a term for moment gradient effect and can be found as $k = 0.2 (f_u/f_y - 1)$, f_u and f_y represents the ultimate and yield strength of longitudinal bars respectively. One should note that the objective of taking the tension shift effect as “ $0.1l_w$ ” is to

accommodate some conservatism for design purposes. For assessment purposes such as predicting the experimental results, " $0.1l_w$ " shall be replaced by " $0.2l_w$ ".

“The plastic hinge lengths have been typically calibrated for the tension side. This becomes evident in the strain penetration term $C d_{bl}$, which is irrelevant if the rotation capacity is limited by compressive strains” (Priestley et al., 1994). Most published plastic hinge equations are therefore not applicable if failure on the compression side of the wall is anticipated.

The studies done in plastic hinge length of RC elements mostly focused on identifying the parameters that impact the spread of plasticity. In other words, Researchers try to calibrate their estimations with the experimental results in order to obtain the values for A,B,C. However some studies investigated the stress-strain properties of the steel and the effect of axial compression, which also have a significant influence on the length of plasticity zone. In case of walls, when subjected to some axial compression, the length over which the longitudinal reinforcement yield is reduced and so does the plastic hinge length (Paulay and Uzumeri, 1975:596-597).

Wallace and Moehle (1992) studied to establish an analytical procedure that determines whether a special boundary element in RC walls is required or not. The outcome of their research suggested that the plastic hinge length varies between $0.5l_w$ and $1.0l_w$ (Wallace and Moehle, 1992: 1633-1634).

Panagiotakas and Fardis (2001) carried out tests on RC members under uniaxial bending, with or without axial loads, to derive expressions for deformations at yielding and failure, as a means of specimen geometric and material properties. The results reported was obtained from 1012 specimens including beams, columns and walls (Panagiotakos and Fardis 2001: 135-148). They developed the following expressions to estimate the plastic hinge length:

$$l_p = 0.12 h_w + 0.014f_y d_{bl} \quad (2.22)$$

The above expression was given for cyclic loading. For monotonic loading, they suggested to take $1.5 l_p$. Authors also stated that if slippage is not expected, the strain penetration term needs to be neglected.

Thomsen and Wallace (2004) conducted laboratory tests of slender RC shear walls with rectangular shaped and T-shaped cross sections with moderate transverse

reinforcement in boundaries. They reported values for the four walls (Thomsen and Wallace 2004: 618-630). The authors compared the experimental and analytical strain profiles, considering plastic hinge lengths of $0.33l_w$, $0.5l_w$ and $0.67l_w$ and determined that plastic hinge lengths between $0.33l_w$ and $0.5l_w$ yields the best agreement between results.

Bohl and Adebar (2011) proposed an expression for l_p based on the results of nonlinear finite-element analysis, which was as a function of wall length, moment-shear ratio, and axial compression ratio the expression:

$$l_p = 0.2 l_w + 0.05 h_w (1 - 1.5P/A_g f'_c) \quad (2.23)$$

Where P is the axial load on the section and A_g is the cross-section area of the wall. The expression is mainly derived from equation 2.20 with proposed changes which aim to take into account the impact of the axial load ratio on the moment gradient contribution to the spread of plasticity i.e the plastic hinge length. It suggests that the presence of relatively high axial load on the section limits the propagation of plasticity zone over a reduced length.

Kazaz et al (2013) stated that “The absence of complete experimental data due to limitations in the experimental measurements and setups, and accuracy problems in section-based moment-curvature analysis procedures in predicting RC response under varying stress conditions, legitimized this study to investigate the distribution of strains and curvatures along the height of the wall at the base. A well-calibrated finite-element modeling tool was used to investigate the plastic hinge analysis method and its components, such as the plastic hinge length, plastic curvature and rotation, and flexural and shear displacements for shear walls.”

They have conducted a parametric study taking into account the wall length (l_w) and height (h_w), as well as the wall aspect ratio (h_w/l_w), axial load level ($1.5P/A_g f'_c$), and the amount of boundary element longitudinal reinforcement (ρ_b), which targets to determine the plastic hinge length (l_p) which is derived from a prediction equation that can be used in the plastic hinge analysis of cantilever walls.

$$l_p = 0.27 l_w (1 - P/A_g f'_c) (1 - \rho_{sh} (f_{yh}/f_c)) (h_w/l_w)^{0.45} \quad (2.24)$$

Where ρ_{sh} and f_{yh} is the reinforcement ratio and the yield strength of transverse reinforcement respectively. In above equation, the term h_w/l_w accounts for the failure mode of RC shear wall and in a respect, represents the normalised shear stresses (Kazaz 2013). They also tried to transform their expression to a similar form of equation 2.20 which leads to :

$$l_p = 0.143l_w + 0.072 h_w \quad (2.25)$$

A parametric study was performed by Massone and Alfaro (2016) in which the following parameters; the axial load ratio $P/A_g f'_c$, the wall length l_w , the wall height h_w , the amount of the boundary reinforcement, the hardening ratio and the drift levels were varied. From the results of numerical analysis, a regression analysis was conducted which modified the expression of Bohl and Adebar (2011) for the plastic hinge length:

$$l_p = 0.2l_w + 0.05h_w(1 - 1.5P/A_g f'_c) 5.1 \Delta_p^{0.3} \quad (2.26)$$

Where Δ_p is the plastic drift ratio which is preferred over total drift ratio. The other terms have been explained in the previous section.

Various expressions were proposed for the calculation of plastic hinge length in Design Codes and Provisions. For RC components, equivalent plastic-hinge length can be simply assumed to be equal to one-half the member depth, $l_p = 0.5l_w$, where l_w is the wall length (Park and Paulay 1975). ASCE/SEI 41 (ASCE 2007) states that for analytical models of shear walls and wall segments, the value of l_p shall be set equal to 0.5 times the flexural depth of the element, but less than 1-story height for shear walls and less than 50% of the element length for wall segments. The flexural depth is recommended to be $0.8l_w$ in Eurocode 8 (European Committee for Standardization (CEN) 2005). The expression proposed for the plastic hinge length in Eurocode 8 reads as:

$$l_p = \frac{L_V}{30} + 0.2l_w + 0.11 \frac{f_y d_{bl}}{\sqrt{f'_c}} \quad (2.27)$$

where L_V is the shear span (moment-shear ratio, M/V), d_{bl} is the (mean) diameter of the tension reinforcement, and f_y and f'_c is the yield stress of the longitudinal reinforcement and compressive strength of the concrete in MPa units, respectively.

Table 2.2: The state of the art plastic hinge length proposals.

The Plastic Hinge Length	Reference
$l_p = 0.08h_w + 0.022f_y d_{bl}$	Priestley and Paulay 1992
$l_p = 0.2l_w + 0.044h_w$	Priestley and Paulay 1992
$l_p = kh_w + 0.2l_w 0.022f_y d_{bl}$	Priestley et al. 2007
$0.5l_w$	Wallace and Moehle 1992
$l_p = 0.12h_w + 0.014f_y d_{bl}$	Panagiotakos and Fardis 2001
$0.33l_w$	Thomsen and Wallace
$l_p = 0.2l_w + 0.05h_w(1 - 1.5P/A_g f'_c)$	Bohl and Adebar 2011
$l_p = 0.143l_w + 0.072 h_w$	Kazaz et al. 2013
$l_p = 0.2l_w + 0.05h_w(1 - 1.5P/A_g f'_c) 5.1 \Delta_p^{0.3}$	Massone et al. 2016
$l_p = 0.27 l_w(1 - P/A_g f'_c)$	Kazaz et al.
$(1 - \rho_v (f_{yv}/f'_c)) (h_w/l_w)^{0.45}$	
$l_p = \frac{L_V}{30} + 0.2l_w + 0.11 \frac{f_y d_{bl}}{\sqrt{f'_c}}$	Eurocode 8 CEN 2005



3. LITERATURE REVIEW ON SHEAR WALL TESTS

3.1 Oesterle et al. (1976-1979)

Oesterle et al. (1976- 1979) reported the results of 16 isolated walls having rectangular, barbell and flanged shaped cross-sections, which were conducted as part of a comprehensive experimental program. Test specimens were approximately 1/3 scale representation of full-size models. The main variables investigated through the experimental program were; the shape of cross-sections, the amount of main flexural reinforcement, the amount of transverse reinforcement at the boundaries and also one specimen was subjected to monotonic loading as to study the influence of loading regime. All test specimens were 75 in. (1905 mm) long and 15 ft. (4572 mm) tall with a 4 in. (102 mm) thick web (Figure 3.1). Design material properties for all specimens set to be identical. Design yield strength for reinforcing bars was $f_y = 60 \text{ ksi}$ (414 MPa) and design nominal concrete compressive strength was $f'_c = 6 \text{ ksi}$ (41.4 MPa). The experimental setup is shown in Figure 3.2.

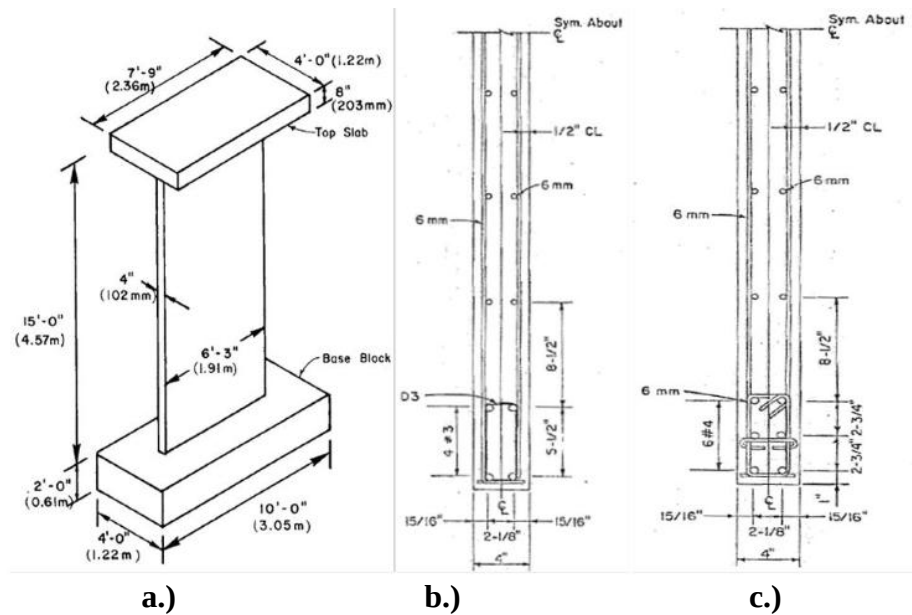


Figure 3.1 : Oesterle et.al. (1976) rectangular walls- a.) specimens dimensions b.) R1 specimen cross-section c.) R2 specimen cross section.

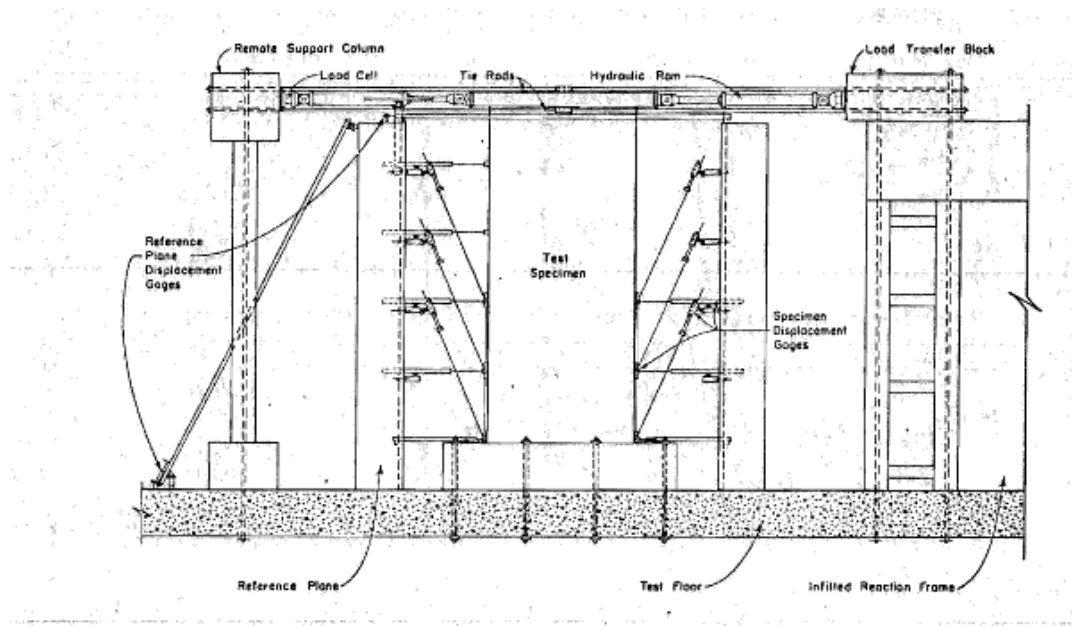


Figure 3.2 : Test Setup.

3.1.1 Test observations

3.1.1.1 R1

- Flexural cracking was first observed in cycle 4
- First yielding occurred in cycle 10
- The maximum load was measured in cycle 16 at a 50.8 mm deflection.
- The first buckling occurred in cycle 20. The outer bars in the compression end of the wall buckled at a location 15 in. above the base. A crack width of 0.05 in. had been previously measured when this end was in compression. Subsequent to buckling of the two bars, ten additional bars buckled during the test.
- First bar fracture took place in cycle 26. The fractured bars were the two that buckled first. Six and one-half cycles were applied. Six and a half cycles were applied between first buckling and fracture. Subsequent to the first two bars fractures, nine additional bars fractured during the test. Each bar fracture was associated with a drop in the load resisted by the specimen.

- During the latter load increments, concrete in the compression zone segmented into large pieces. However since the wall was lightly reinforced in flexure, the concrete did not crush.
- The specimen sustained at least 80% of the maximum measured load through 13 complete inelastic cycles. The last inelastic loading increment in which the load was sustained at or above 80% of the maximum for all 3 cycles was at 3 in. (76.2mm)
- The last stable Cycle was 25 and the maximum observed deflection during this cycle was -4.06 in (103.12 mm).
- Note that the reported ultimate deformation was observed in cycle 25 while the first bar fracture occurred in cycle 26. Failure mode was said to be inelastic bar buckling
- Load-deflection plot of R1 is shown in Figure 3.3.

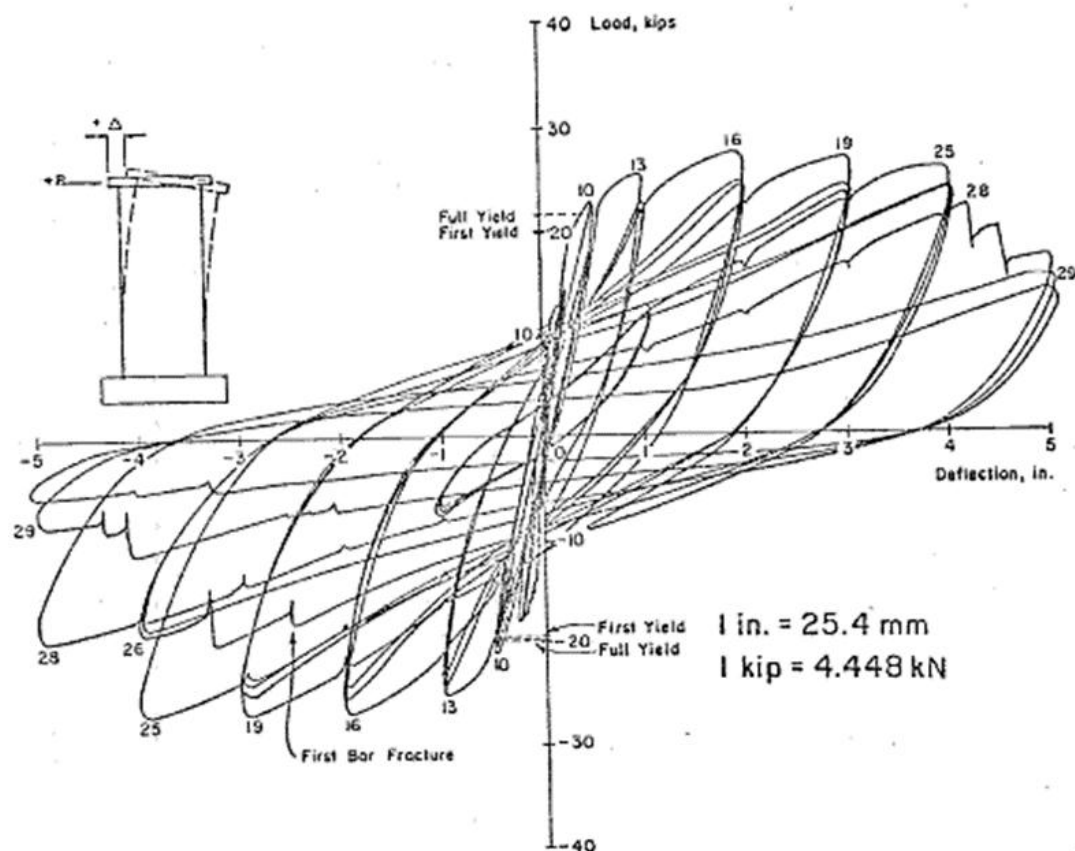


Figure 3.3 : Oesterle et al. (1976) – Load-deflection plot of R1.

3.1.1.2 R2

The out of plane displacement of the compression zone was caused by alternate tensile yielding of flexural reinforcement. As the specimen loaded beyond yield in one direction and then unloaded, permanent deformations remained in the tensile steel. As the load was reversed, the compression zone was supported across numerous cracks only by the steel. This steel had sustained several previous inelastic strain reversals. Therefore the section stiffness of the compression zone for axial load was decreased considerably and an out of plane displacement resulted. The fact that lateral support was not present during a major portion of the test probably led to premature strength loss in specimen R2.

- First yielding was observed in cycle 19
- During cycle 28 bowing of the compression end was observed. The boundary element was 6.4 mm out of plane. Although this bowing progressed further with each cycle, the load bearing capacity remained stable.
- After cycle 32 the boundary element was 76.2 mm out of plane. The test was stopped and lateral bracing was added to the test setup.
- The maximum load was measured in cycle 34 at a 5 in deflection.
- In cycle 35 a large out of plane displacement of the compression zone was observed which accompanied by the decrease of load capacity. This cycle is reported to be the last stable cycle and the ultimate deflection measured was 5.25 in. (127 mm).
- Considerable crushing and loss of load carrying capacity.
- Ultimate deformation was observed in cycle 35 subsequent to an extensive boundary element out of plane deflection which also caused the degradation of the maximum strength.

3.1.1.3 B1

- The first indication of crushing of the concrete at the base of the wall occurred in Cycle 22, also during Cycle 22, first flexural bar buckling occurred.
- After first buckling of flexural bars, the boundary elements started to deteriorate. The outer shell spalled off and small pieces would fall out of the

core when the element was in tension. However, the load carrying capacity did not reduce for several additional cycles.

- The maximum load measured in Cycle 28 at a 4-in deflection. The end hooks of the horizontal steel started to open
- First bar fracture occurred in Cycle 31. The bar that had buckled first was the first to fracture. Subsequently, 3 more bars fractured but the loss of strength was not above 20%.
- During Cycle 33, the load capacity dropped by 30% which was obviously the result of bar fractures.
- Last stable cycle was 32. The failure mode was reported to be inelastic bar buckling (Figure 3.4).

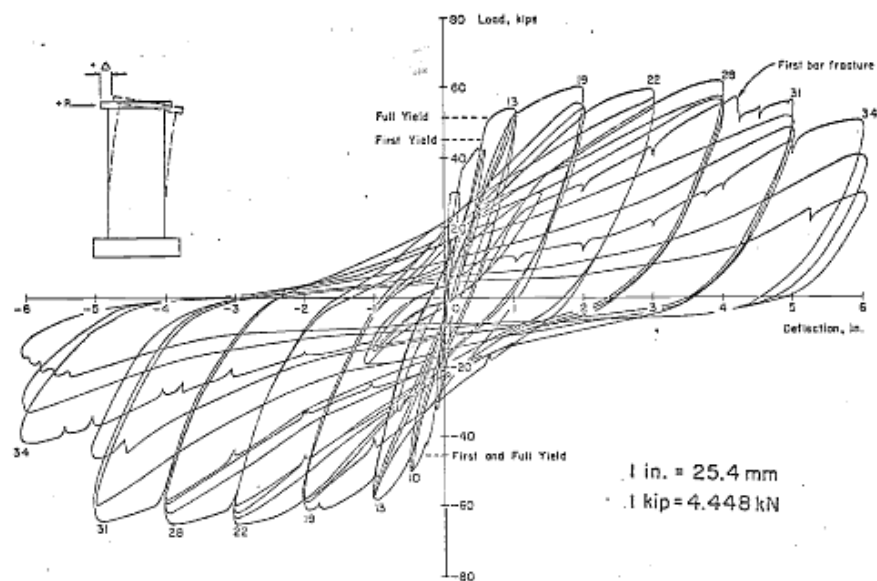


Figure 3.4 : Oesterle et al. (1976) – Load deflection plot of B1.

3.1.1.4 B3

The behaviour of B3 was very similar to the behaviour of B1 through the first 21 cycle. However, whereas flexural buckling of vertical bars occurred in cycle 22 for B1, no significant distress was observed in the boundary elements of B3 until Cycle 38. Significant crushing and grinding progressively deteriorated the web concrete after

cycle 28. However, the confinement hoops in the columns contained the boundary element concrete and helped to prevent bar buckling.

- The maximum load was measured during cycle 34
- In cycle 39, the compression boundary element was shear through, no significant loss of capacity was reported.
- In cycle 39, a vertical bar fractured at the base of the wall while loading the specimen to 7 in. The fractured bar still had concrete and showed no evidence of previous bar buckling. During this cycle, a bar fractured in the opposite column which had been buckled prior to fracture. The load bearing capacity remained above 85% of the maximum. In subsequent cycles, buckling was noted in 7 main flexural bars and 5 of those fractured $\frac{1}{2}$ or 1-1/2 cycles after buckling.
- Last stable cycle was 39 (Figure 3.5).

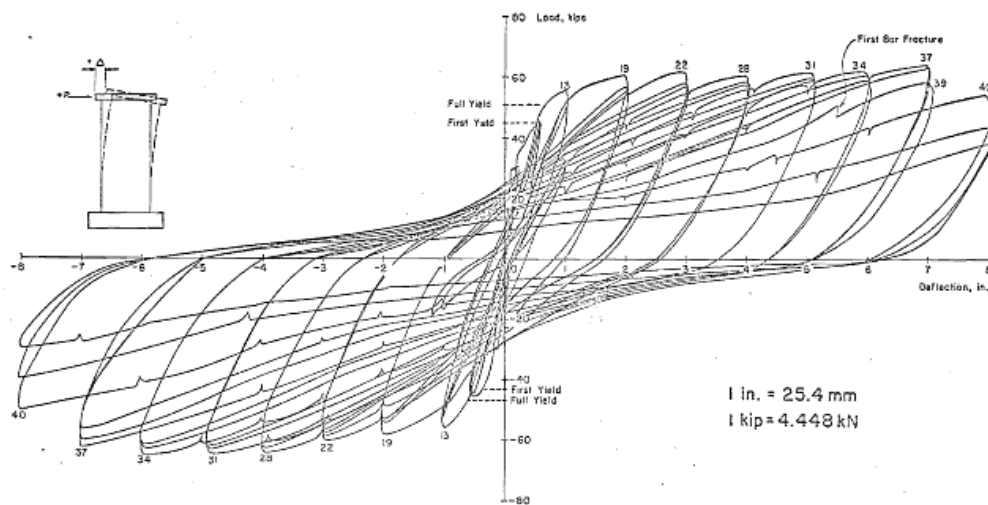


Figure 3.5 : Oesterle et al. (1976) – Load-deflection plot of B3.

Buckling did not result in loss of ultimate capacity however it had a critical impact on bar fracture. Generally previously buckled bars fractured, which caused the lateral load capacity to drop by 20%. Thus the failure mode was reported as inelastic bar buckling.

The Force-displacement hysteresis is shown in Figure 3.5.

3.2 Dazio et al. (2009)

Dazio et al. (2009) conducted tests on wall panels representing common design and construction practice in Central Europe, which were designed to resist moderate earthquake actions. The test series comprised six large scale rectangular shaped cantilever walls tested under quasi-static loading (Figure 3.8). All specimens were half scale representation of the lower portion of a six storey wall. The main objective of the test series were to investigate the impact of the ductility properties of steel reinforcement and the amount of the longitudinal bars. All test specimens were 1905 mm (78.7 in) long and 4030 mm (159 in) tall with a 150 mm (5.9 in.) thick web (Figure 3.6). Average test day yield strength for reinforcing bars varied between $f_y = 70 \text{ ksi}$ (485 MPa) and 95 ksi (656 MPa) and Average test day concrete compressive strength varied between $f'_c = 5.6 \text{ ksi}$ (38.3 MPa) and 6.6 ksi (45.6 MPa). The experimental setup is shown in Figure 3.6. The Test units were labelled WSH1 to WSH6. The ratio of constant axial load applied ranged between $0.05 A_g f'_c$ and $0.10 A_g f'_c$.

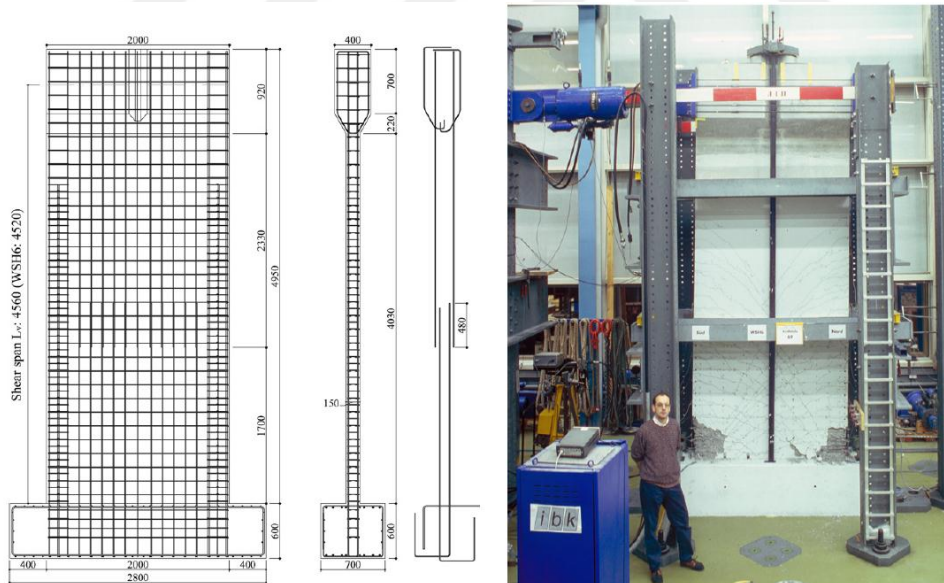


Figure 3.6 : Dazio et al. (2009) – Overall Dimensions and Test Setup .

Cross-sectional geometries and reinforcement layouts of the specimens are shown in Figure 3.7. All specimens had the confined boundaries but one, WSH4. Two types of boundary detailing were preferred for confinement. Type 1 comprised of a perimeter hoop and a single cross tie (WSH1, WSH2, WSH3), the other type consisted an outer hoop and a centre hoop, overlapping style.

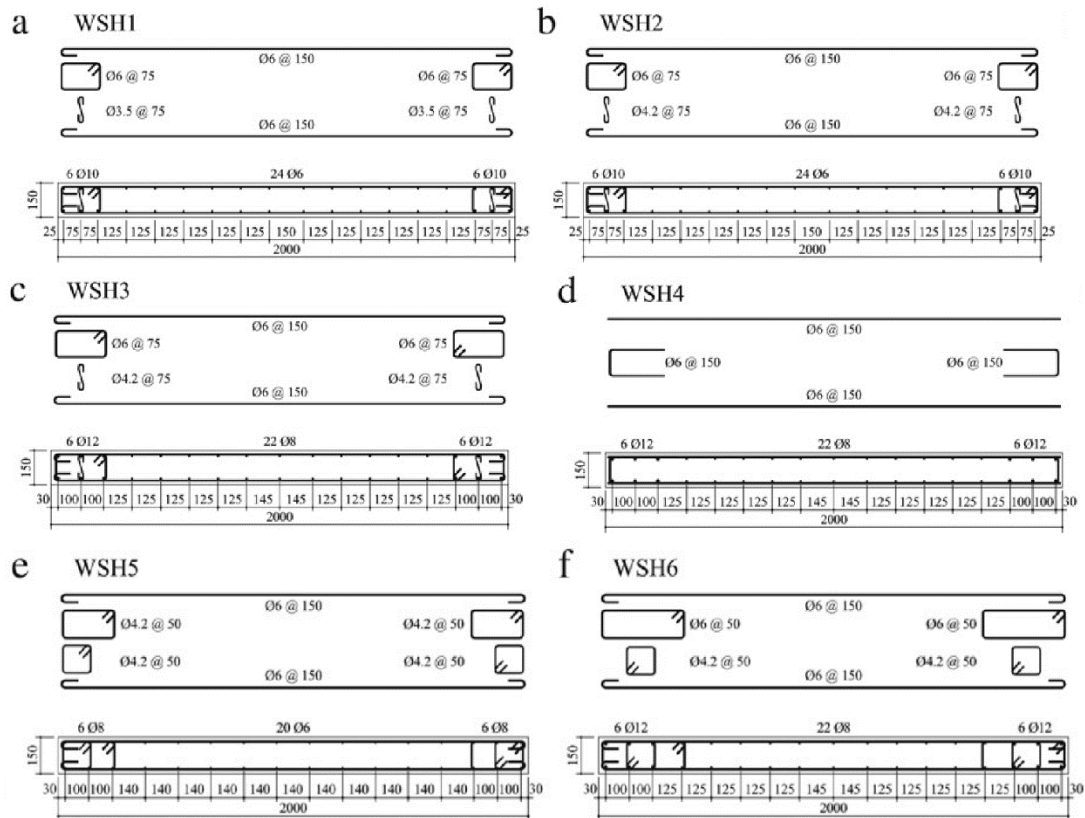


Figure 3.7 : Dazio et al. (2009) – Cross section geometries and reinforcement layouts.

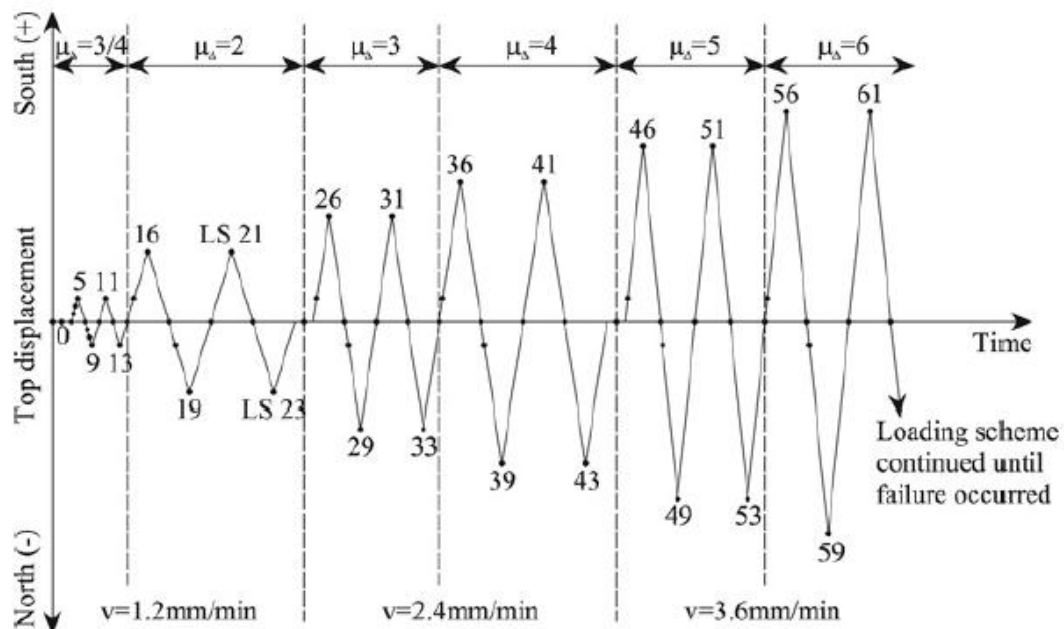


Figure 3.8 : Dazio et al. (2009) – Loading scheme.

3.2.1 Test observations

3.2.1.1 WSH1

The ductility properties of the longitudinal reinforcement was the poorest of all which exhibit no hardening at all and had the smallest strain capacity. As a result, even before the onset of cover concrete spalling, the web vertical bars fractured at a relatively small drift 0.68%. The Ultimate point reported as the instant when three of the innermost D10 mm bars ruptured at the north boundary which caused 21% drop of the maximum strength. It is noteworthy that the bars had not been buckled prior to tensile rupture. The force-displacement hysteresis of WSH1 can be found in Figure 3.8.

3.2.1.2 WSH2

The reinforcement layout was the same as for WSH1 but the ductility properties of the bars were better which delayed the drift 1.16% at which the fracture of web longitudinal bars were first observed. Although this was a drift that was larger than the ultimate drift of WSH1, at this instant, cover concrete spalling had not yet been initiated. The ultimate point was associated with the buckling of all six D10 mm bars located at the boundary at a drift of 1.36%. However, the ultimate point can be adopted as the point where all previously buckled bars ruptured at 1.80% drift leading to the peak strength drop to 78%.

3.2.1.3 WSH3

Of all the six specimen, WSH3 was the one to satisfy all the ductility requirements of Central European Standards in terms of the ductility properties, contents and detailing of reinforcement bars. As a consequence, no premature fracture/rupture of bars was observed prior to cover concrete spalling. Spalling started at 1.02% drift. As the specimen pushed further, the longitudinal bars became visible exhibiting early signs of buckling. At a drift of 1.79%, one of the corner bars which had previously buckled ruptured. The test was ended at the drift of 2.04% Although the drop in peak strength had not reached 20%.

3.2.1.4 WSH4

Not only the longitudinal reinforcement layout of WSH4 was the same as the one of WSH3, but also the ductility properties of those two specimens were pretty similar

however WSH4 had no confining transverse bars at the boundaries which means the test unit was not designed to meet the ductility requirements. The onset of concrete spalling was observed at a drift of 1.02% .Since the longitudinal bars at the boundaries had not been confined by hoops or hooks, buckling effectively commenced at the same with concrete spalling at 1.02%. Although buckling of the boundary bars took place, the specimen could not resist until the fracture of those buckled bars and the crushing of the unconfined concrete finalized the test at 1.35% drift.

3.2.1.5 WSH5

The moment capacity of WSH5 designed to be same as WSH3 but as the axial load was larger for WSH5, the amount of longitudinal bars needed to be decreased. The spacing of confining bars reduced to 50 mm as to provide the same $s/d_b = 6.25$ ratio as WSH3. Since the web reinforcing bars did not have favourable ductility properties, the fracture of those bars was observed at a drift of 1.01%. Upon load reversals, the rupture of previously buckled boundary bars ended the test at a drift of 1.36%.

3.2.1.6 WSH6

The same axial load as WSH5 applied to the specimen WSH6 which had the largest moment capacity of all test units as it had the same longitudinal reinforcement layout as WSH3 and the highest axial load ratio. The transverse reinforcement had been located at a space of 50 mm and the ratio of $s/d_b = 4.17$. Concrete cover spalling initiated at a drift as small as 0.57% which can be attributed the high axial stress demands on unconfined concrete. As a result of smaller s/d_b ratio, the onset of buckling was not observed until the cycles at 1.70% drift. It can be concluded that since the buckling had been delayed, the fracture of main longitudinal bars prompted by buckling did not occur whereas the high axial loads imposed relatively higher transverse strain demands on confining bars, which in return end up with the fracture of stabilising bars ending the test at a drift of 2.07%.

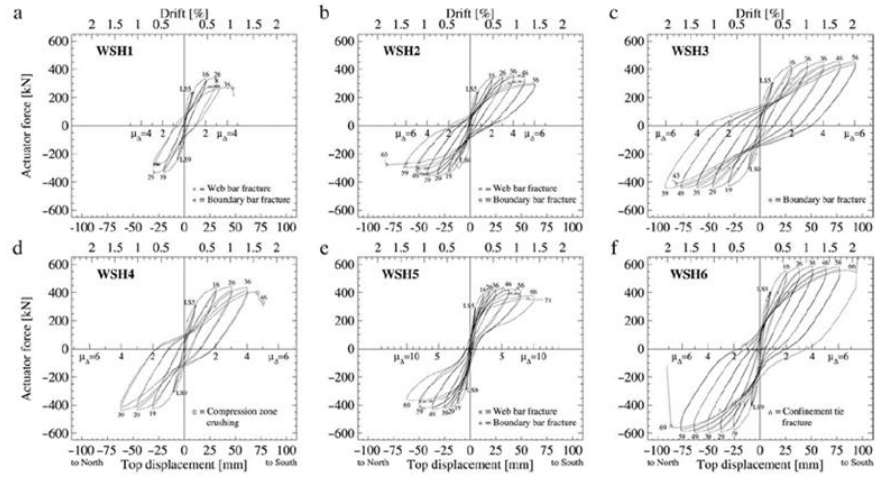


Figure 3.9 : Dazio et al. (2009) – The force-displacement hysteretic curves of the all six specimen.

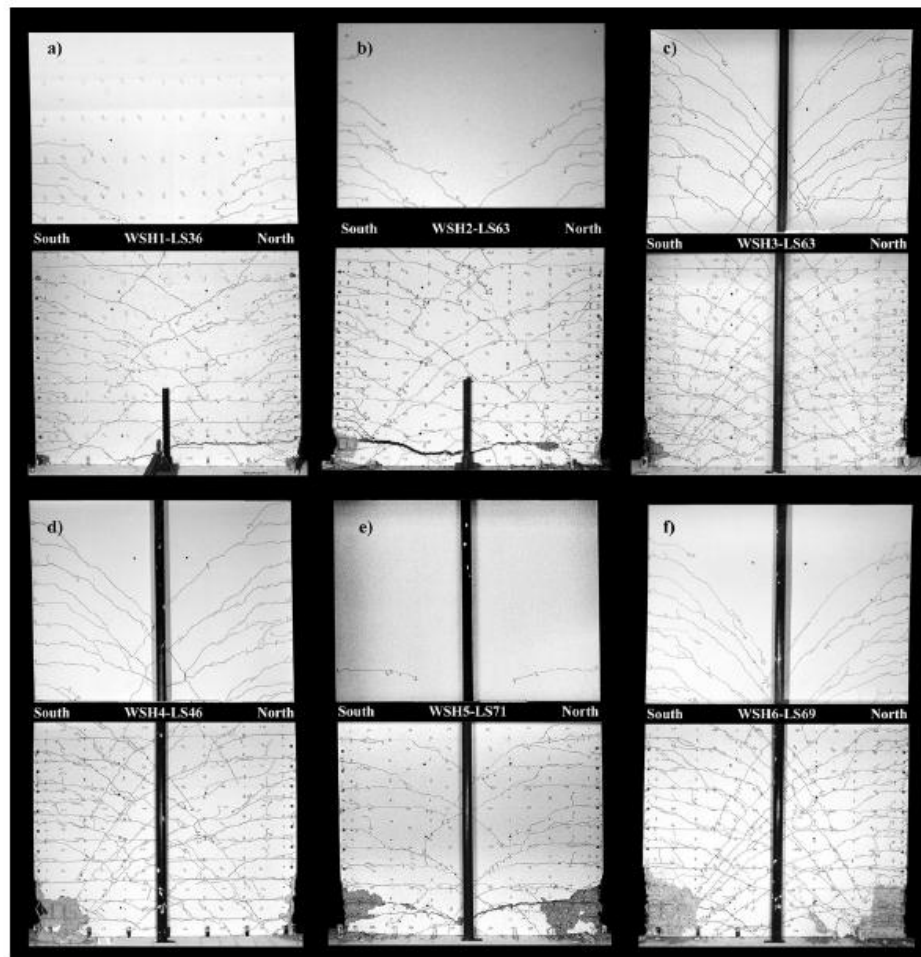


Figure 3.10 : Dazio et al. (2009) – Final Damage State of the all six specimen.

3.3 Segura (2017)

Segura (2017) conducted a series of laboratory tests comprised of 7 one-half scale, ACI 318-14 compliant wall specimens, representing of the botom 1.5 stories of an eight story cantilever wall. The specimens were subjected reversed cyclic lateral loads and constant axial load. The first phase of the experimental program mainly focused on identifying probable deficiencies in current provisions. The primary variables investigated in the first phase was the configuration of boundary longitudinal reinforcement, the amount and the detailing of boundary transverse reinforcement and compression block depth (neutral axis position). For the second phase, the test units were designed with either thicker cross-section, improved confining details at boundaries, or both. All seven wall panels were 2286 mm (90 in.) long and 2134 mm (84 in.) tall with a 150 mm (5.9 in.) (Figure 3.12). Web thickness of wall specimens ranged between 6 in.(152mm) and 9 in. (229). Design yield strength for reinforcing bars was $f_y = 60 \text{ ksi}$ (414 MPa) and design nominal test day concrete compressive strength was $f_c' = 5.0 \text{ ksi}$ (34.5 MPa). Average test day yield strength for longitudinal reinforcing steel bars varied between $f_y = 65.8 \text{ ksi}$ (454 MPa) and 83.9 ksi (578 MPa). Average test day concrete compression strength differed between $f_c' = 5.18 \text{ ksi}$ (35.8 MPa) and $f_c' = 7.04 \text{ ksi}$ (48.6 MPa) . The experimental setup is shown in Figure 3.11. The Test units were labelled WP1 to WP7. The ratio of constant axial applied ranged between 270 kip and 405 kip .

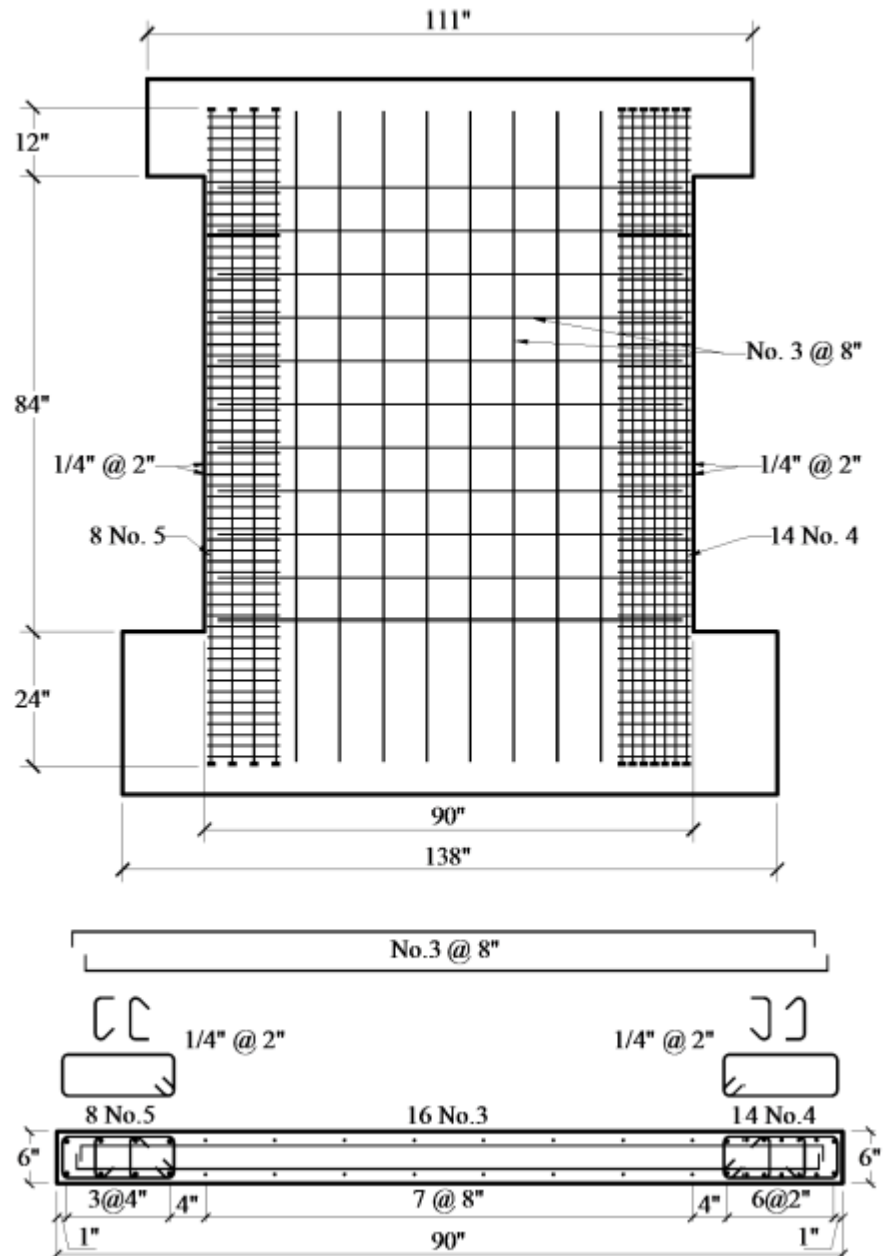
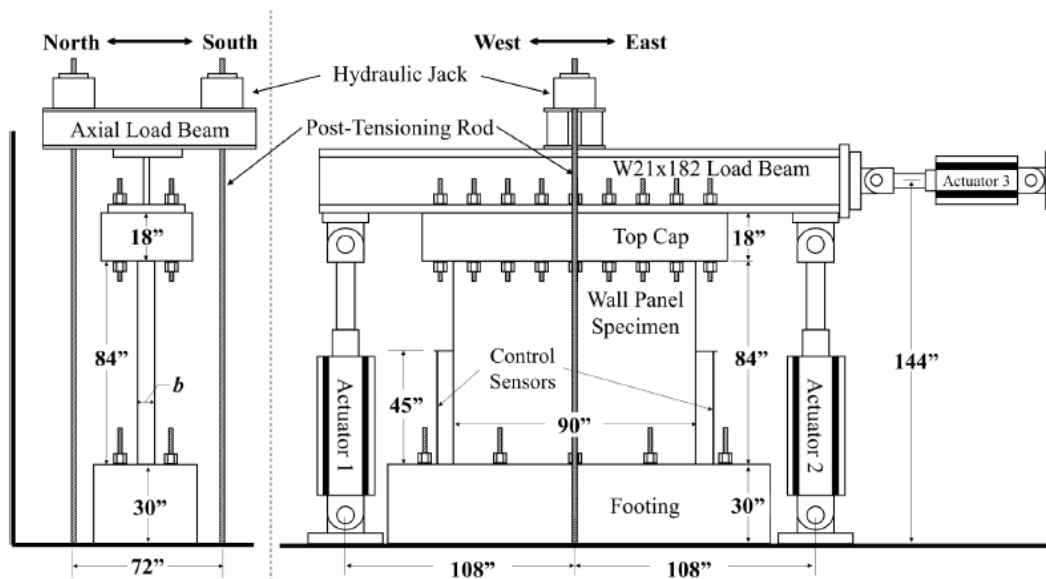


Figure 3.11 : Segura (2017) – Specimen WP1 reinforcement layout and overall dimensions.



Specimen/ Boundary		Primary Test Variable	<i>b</i>	Axial Load	Confinement Detail	<i>A_{shl}</i> / <i>A_{ACT}</i> *	<i>A_{shl}</i> / <i>A_{ACT}</i> **	<i>s/d_b</i>	<i>c/l_w</i> *	<i>c/l_w</i> **
			in.	kip						
WP1	West	Reference Specimen	6.0	270	HCT 90°-135°	1.02	0.80	3.2	0.21	0.23
	East							4.0	0.20	0.22
WP2	West	<i>A_{sh}</i>	6.0	270	HCT 90°-135°	1.24	1.00	3.2	0.21	0.22
	East							4.0	0.20	0.21
WP3	West	<i>s/d_b</i>	6.0	270	HCT 90°-135°	1.21	1.38	6.0	0.21	0.21
	East					1.09	0.87		0.20	0.20
WP4	West	<i>c/l_w</i>	6.0	270	HCT 90°-135°	1.23	0.91	3.2	0.30	0.30
	East				---	---	---	---	0.06	0.07
WP5	West	Boundary Detail	7.5	337.5	CTR	2.05	1.57	3.2	0.20	0.17
	East		6.0	270					0.21	0.19
WP6	West	<i>b</i>	7.5	337.5	CTR	1.64	1.30	β.2	0.20	0.18
	East				HCT 135°-135°	1.02	0.90			
WP7	West	<i>b</i>	9.0	405	CTR	1.64	1.24	3.2	0.20	0.18
	East				HCT 135°-135°	1.02	0.85			
<i>A_{shl}</i> : area of transverse reinforcement in direction perpendicular to wall length; <i>A_{ACT}</i> : area of transverse reinforcement required by ACI 318-14 HCT: single outer hoop and cross-ties (90° or 135° cross-tie hook indicated); CTR: continuous transverse reinforcement * Determined using nominal material properties; ** Determined using test day material properties										

Figure 3.14 : Segura 2017 – Test Matrix.

3.3.1.1 Specimen WP1

The specimen was the reference specimen of all seven.

- Horizontal flexural cracking of concrete was first observed at the east boundary at +0.05 rotation.
- Vertical splitting of cover concrete was observed at +0.5 rotation at west edge and at -1.0 rotation at the east edge.
- Through 3 cycles to ±1.0 % rotation no strength loss was observed for the specimen WP1.
- The peak lateral strength was measured at +1.26%. The measured overturning base moment was 1.07 M_N .
- Onset of buckling of the bars were revealed at the northwest and the southwest corner of the walls following concrete cover spalling at 1.5%.
- Opening of 90 degree cross-tie hook was observed at the time cover spalling took place.
- Loading in the positive direction was terminated as very little residual strength remained this was due mostly to the fact that after cover crushing, out-plane displacements increased steadily causing a significant drop in capacity.

- Monotonic loading was applied in the negative direction. At -1.97% rotation, abrupt crushing of core concrete at the east boundary was observed and the all boundary bars became visible. After the removal of loose concrete, it was seen that all three of the hoops at the west boundary had fractured. (Figure 3.15).

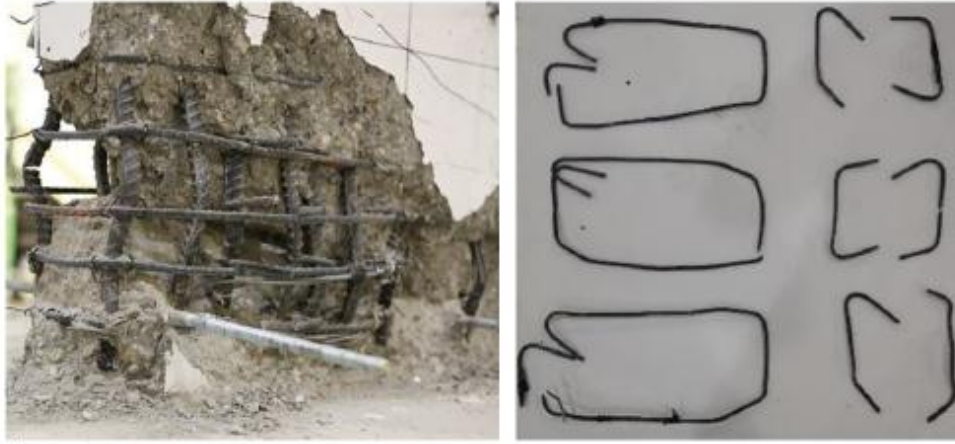


Figure 3.15 : Segura 2017 – WP1 east boundary final damage state.

3.3.1.2 Specimen WP2

Specimen WP2 had the same cross-sectional properties and similar longitudinal reinforcement as WP1 however the quantity of transverse reinforcement at the boundaries was the variable to distinguish WP2 from WP1.

- Horizontal flexural cracking of concrete was first observed at the east boundary at +0.049 rotation and -0.053% rotation at the west boundary.
- The peak lateral strength was measured at +1.35 %. the measured overturning base moment was $1.09 M_N$. Maximum lateral strength in the negative loading was measured at -1.41 % rotation and the corresponding moment was $1.01 M_N$.
- Crushing of the confined region and a portion of the wall web occurred reaching +1.5% rotation. Positive loading was terminated as the instability at the west end followed crushing of concrete.
- Monotonic loading was applied in the negative direction. At -1.78% rotation, abrupt crushing of core concrete at the east boundary was observed. After the removal of loose concrete, it was seen that some of the hoops at the west boundary had fractured. The final damage stage is shown in Figure 3.16.

It is noteworthy to state that although the amount of the boundary transverse reinforcement of WP2 was larger than the one of WP1, WP2 showed no improved displacement capacity comparing to WP1, which can be attributed to the fact that the strain capacity of transverse bars at the boundaries of WP2 might be smaller than WP1.

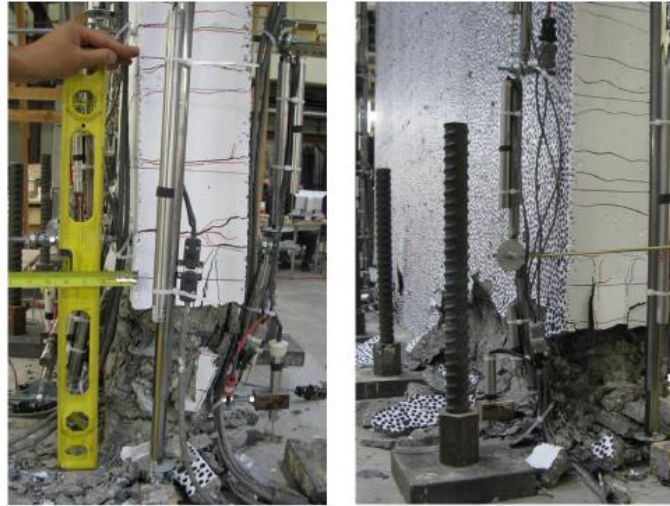


Figure 3.16 : Segura (2017) – WP2 east and west boundary final damage state.

3.3.1.3 Specimen WP3

The primary test variable was the ratio of s/d_b which was 6 for specimen WP3 whereas WP1 for which s/d_b was 3.2. The observed behaviour for WP3 was similar to WP1 during cycles up to 1.00% rotation.

- Horizontal flexural cracking of concrete was first observed at the east boundary at +0.048 rotation and -0.050% rotation at the west boundary.
- The peak lateral strength was measured at +1.38 %. The measured overturning base moment was $1.01 M_N$.
- Buckling of boundary bars was observed at +1.50% rotation at the west boundary and the east boundary bars buckled in the negative loading at -1.50%.
- During the third cycle to +1.5% rotation, crushing of the confined region of the west boundary and a portion of the wall was observed.

- While loading to -1.5% rotation for the third time, three of the eight buckled bars at the west boundary fractured. As loading continued in the negative direction an abrupt crushing of the confined region of the east boundary was observed. The final damage stage is shown in Figure 3.17.



Figure 3.17 : Segura (2017) – WP3 east and west boundary final damage state.

3.3.1.4 Specimen WP4

Specimen WP4 was mainly tested for investigating the impact of compression depth. Axial load and reinforcement detail was designed in a way so that the compression depth was 30% of the wall length at the west boundary whereas for the east boundary, which was constructed with an enlarged t-shaped boundary, the compression depth was only the 6% of the wall length. As can be anticipated, the observed behaviour was completely different in the negative loading and the positive loading. Negative loading referred to the loading when the flange boundary is in compression.

- Horizontal flexural cracking of concrete was first observed at the east boundary at +0.054 rotation and -0.060% rotation at the west boundary.
- The peak lateral strength was measured at +0.91%. The measured overturning base moment was $1.01 M_N$ while the peak strength was measured at -2.91% in the negative loading.
- During the first cycle to +1.5% rotation, at a rotation of %1.32, an abrupt compression failure including concrete crushing and reinforcement buckling. The positive loading was terminated.
- While monotonically loading in the negative direction, minor crushing of cover concrete was observed at -2.5% rotation, at least one bar fractured at the west boundary. As loading continued in the negative direction several west

boundary longitudinal bars fractured in tension. At -3% rotation the peak strength dropped to 80% ending the test. The final damage stage is shown in Figure 3.18.



Figure 3.18 : Segura (2017) – WP4 east and west boundary final damage state.

3.3.1.5 Specimen WP5

Specimen WP5 had different wall thickness along the east and the west half of the wall which was 6 in. (152 mm) thick web for the east half of the wall, and a thicker web 7.5 in (191 mm) for the west half of the wall. Both boundaries had continuous transverse reinforcement detailing and the amount of confining bars was the twice as much as required by ACI 318-14. To assure the similar compression depth for the both end, the applied axial load differed in the positive and the negative loading.

- Horizontal flexural cracking of concrete was first observed at the east boundary at +0.058 rotation and -0.048% rotation at the west boundary.
- The peak lateral strength was measured at 2.02 % for the negative and positive loading. The measured overturning base moment was $1.21 M_N$.
- No drop in capacity was observed during the two complete cycles to +/-2% rotations. crushing and spalling of concrete was observed but no damage was evident for core concrete at either boundary. Slight buckling of main bars was observed.
- When the test unit was pushed further up to +3% rotation, three of the previously buckled bars at the east boundary ruptured that caused an 18% drop from peak strength.

- While loading in the negative direction to -3% rotation, the east boundary began to slide out-of plane, this was due mostly to the eccentricity caused by rupture of bars and asymmetric crushing/spalling of concrete. A lateral displacement of 2.75 in. was measured when the loading halted at -3% rotation. However loading in the positive direction went on till +4% rotation, all the longitudinal bars fractured at the west boundary and the overturning demand was too low to impose any kind of compression demands at the west boundary. The test was stopped. The final damage stage was demonstrated in Figure 3.19.

The west boundary had yet not been significantly damaged even at the end of the test owing to the reason that the boundary elements were satisfactorily detailed as a means of confinement to meet the imposed transverse demands and were able to delay the concrete crushing of confined region.

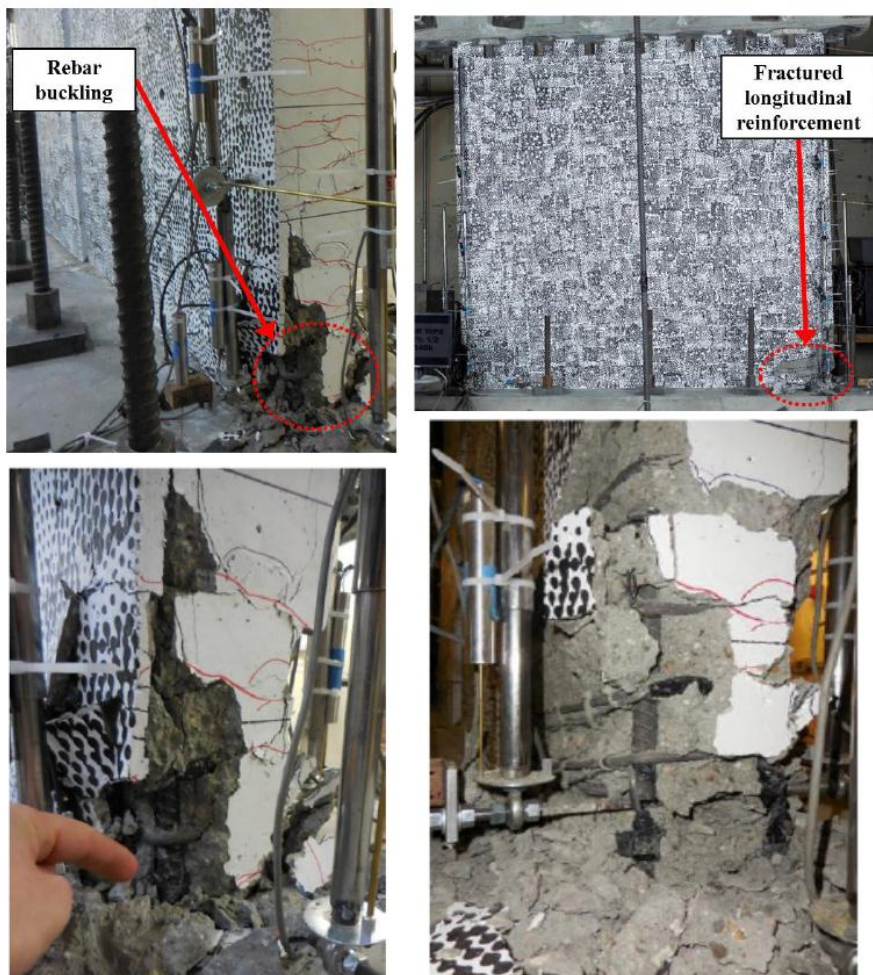


Figure 3.19 : Segura (2017) – WP5 east boundary final damage state.

3.3.1.6 Specimen WP6

The primary variable tested for WP6 was the web thickness along with the secondary variable of 135-degree cross-ties in the middle with an outer hoop.

- Horizontal flexural cracking of concrete was first observed at the east boundary at +0.047 rotation and -0.037% rotation at the west boundary.
- The peak lateral strength was measured at + 2.99% for the positive loading and -2.03 for the negative loading. The measured overturning base moment was $1.20 M_N$.
- No signs of profound strength loss was observed during two complete cycles up to +/-2 rotation.
- Onset of buckling for the west boundary longitudinal bars were reported to be at +2% rotation. While loading to -2% rotation, one of the bars located at the southwest corner fractured. Visible damage at the east boundary was only limited to minor crushing/spalling of cover concrete (Figure 3.20).
- During cycles up to -3% rotation, several more longitudinal bars at the west boundary fractured causing the peak strength drops to 71%. The east boundary confined boundary crushed prior to reaching -3% rotation. While attempting to reload to +3%, out of plane sliding was appeared at the west boundary.
- While loading to +3% rotation, three outer bars at the east boundary ruptured.

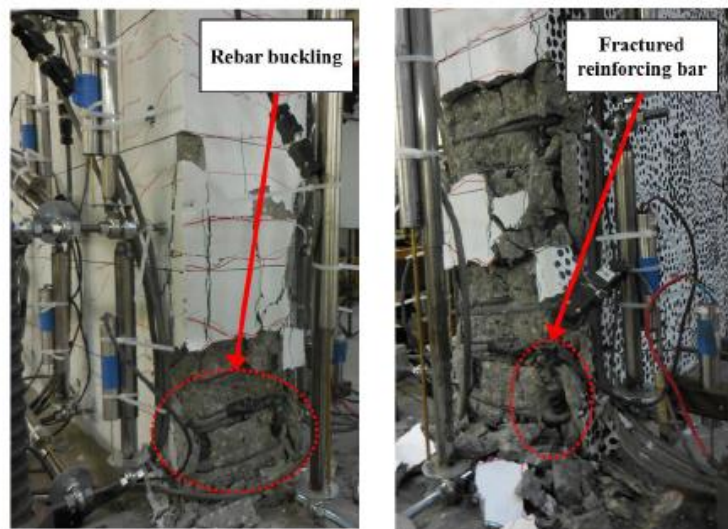


Figure 3.20 : Segura (2017) – WP6 west boundary final damage state.

3.3.1.7 Specimen WP7

Test unit WP7 had the thickest web of all. The other features of test specimen WP7 was pretty similar to those of WP6.

- Horizontal flexural cracking of concrete was first observed at the east boundary at +0.051 rotation and -0.047% rotation at the west boundary.
- The peak lateral strength was measured at -1.79 for the negative loading. The measured overturning base moment was $1.15 M_N$. For the loading in the positive direction, the maximum capacity of the base moment was measured at +2.67 rotation, which was $1.24 M_N$.
- No signs of profound strength loss were observed during two complete cycles up to +/-2 rotation.
- Onset of buckling for the east boundary longitudinal bars were observed at a rotation of -2%. As the rotation reach to -3%, severe buckling of all three outer bars at the east boundary was observed. At -2.98 rotation, abrupt crushing of east boundary occurred while the fracture of confining hoops were onserved.
- After terminating the test in the positive loading due to the failure of the east boundary, the specimen was monotonically pushed further in the positive loading up to +4.5% rotation. All the east boundary bars fractured leading to a residual strength of 44%. While loading to +3% rotation, three outer bars at the east boundary ruptured Figure 3.21).

The west boundary remanied stable under compression even up to +4.5% rotation.



Figure 3.21 : Segura (2017) – WP7 west and east boundary final damage state.

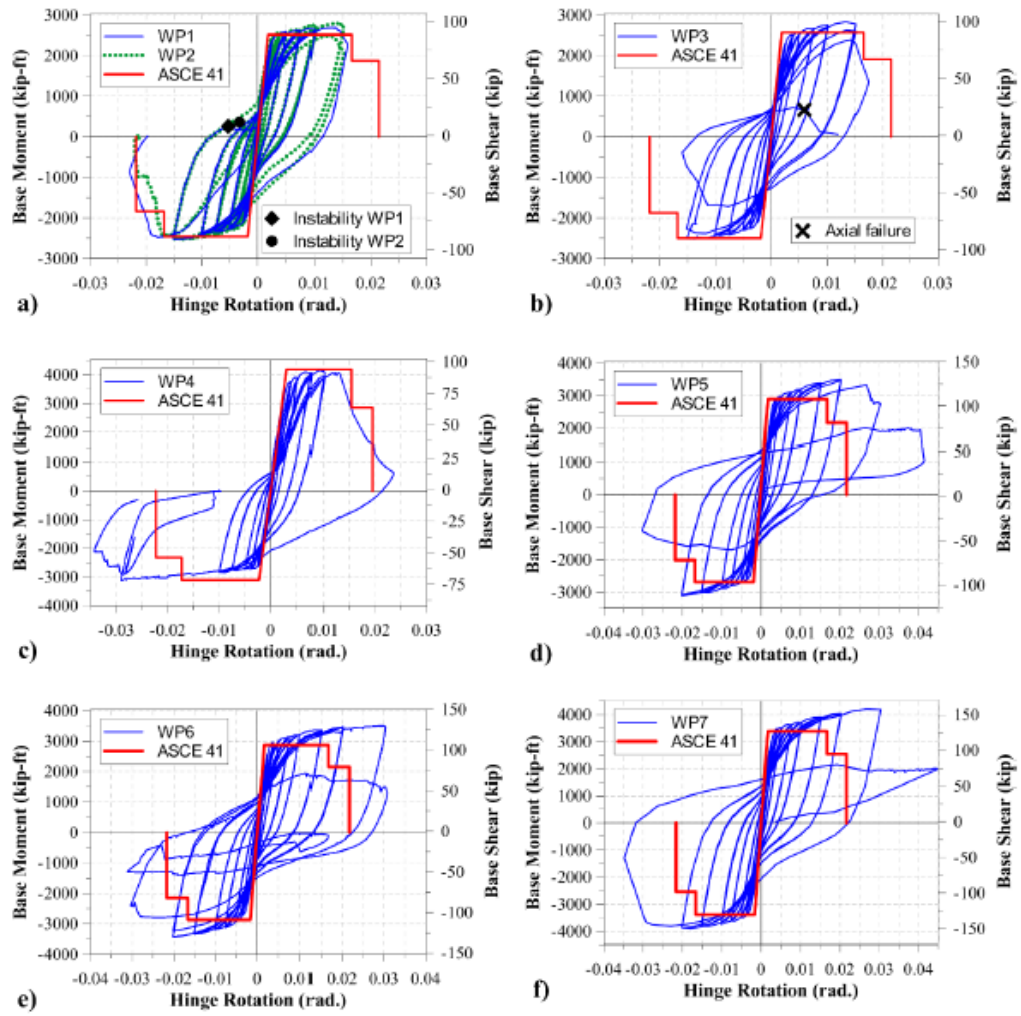


Figure 3.22 : Segura 2017 – Base Moment vs Hinge rotation.

3.4 Thomsen and Wallace (1995, 2004)

Wallace (1994, 1995) proposed a displacement-based design methodology which then validated against a test program including two rectangular shaped and two T-shaped walls. Overall dimensions and test setup is shown in Figure 3.23. All of the specimens were 12 ft (3658mm) tall, 4 in. (102mm) thick web with a 4ft. (1219 mm) long. Design yield strength for reinforcing bars was $f_y = 60 \text{ ksi}$ (414 MPa) and design nominal test day concrete compressive strength was $f'_c = 4.0 \text{ ksi}$ (27.4 MPa). The same axial load of $0.10 A_g f'_c$ were applied to the all test units. The cross-sectional geometry and reinforcement details are shown in Figure 3.24.

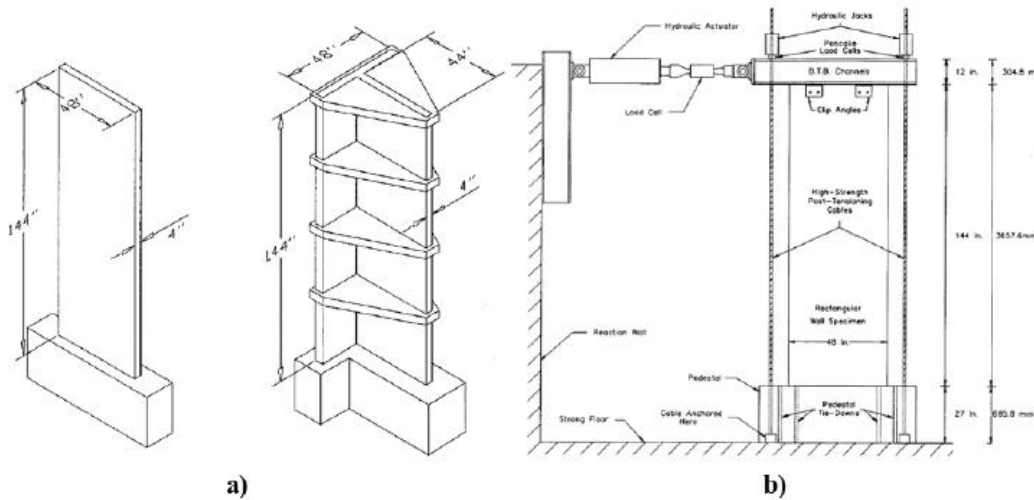


Figure 3.23 : Thomsen and Wallace (1995) – a) Overall geometry and b) Test setup.

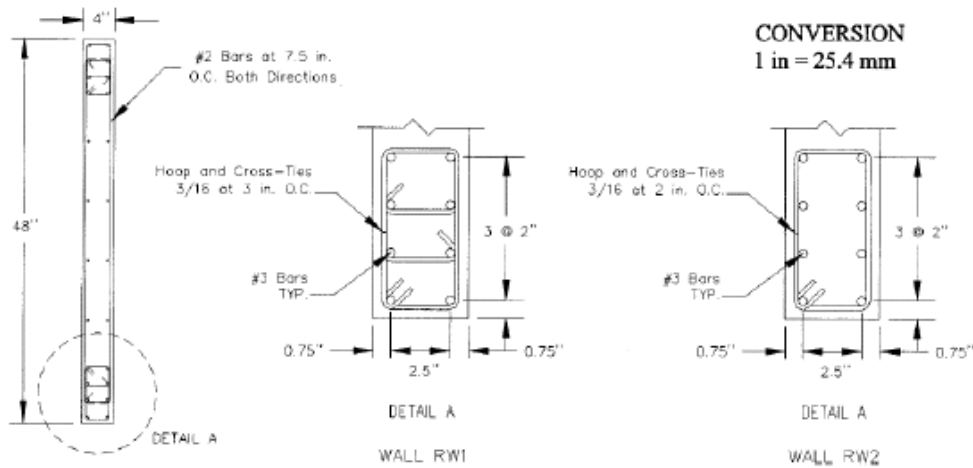


Figure 3.24 : Thomsen and Wallace (1995) – Cross-section details of rectangular walls (RW1 and RW2).

During the first cycle up to 2% drift, buckling of the outermost longitudinal reinforcing bars appeared at each end of RW1. The failure of the specimen at which peak strength dropped significantly at a drift of 2.5% was due to the buckling of main longitudinal bars. Since that transverse bars of boundaries located at a rather larger space having $s/d_b = 8$, inelastic bar buckling was not an unforeseen failure for this specimen. Specimen RW2 exhibit almost the same behaviour as RW1 until two complete cycles at 2% drift. However RW2 was able to endure even after two complete cycles up to 2.5% drift which was due mostly to the fact that closer spacing of the confining bars provided a delayed buckling for RW2. The failure of RW2 was reported to be crushing of confined concrete (Figure 3.25).

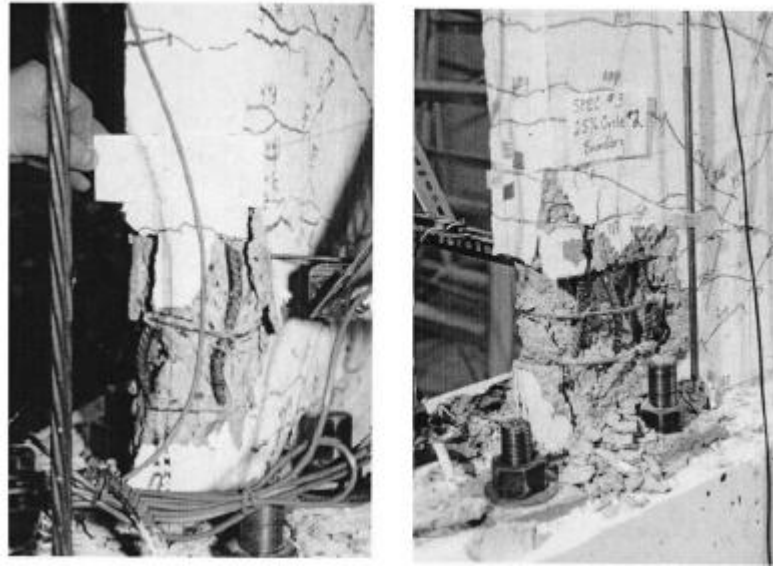


Figure 3.25 : Thomsen and Wallace (1995) – Final damage stage of RW1 and RW2.

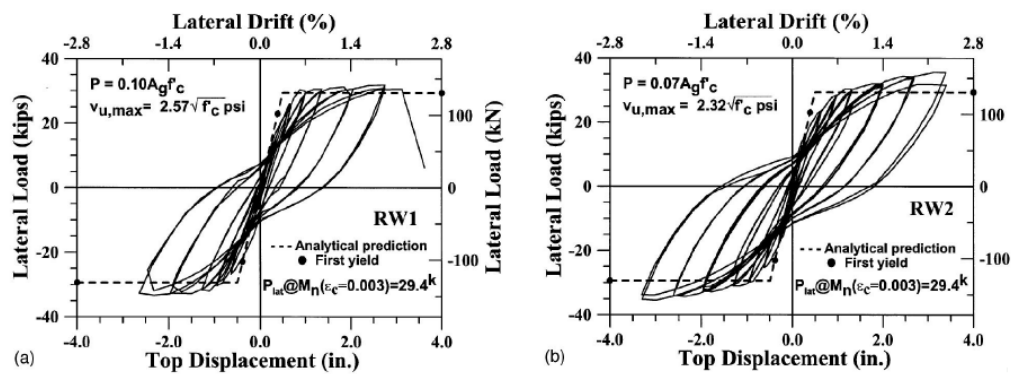


Figure 3.26 : Thomsen and Wallace (1995) – Load–deformation responses.

3.5 Birely (2011), Lowes et al. (2012)

The planar wall test program sought to test large scale specimens that represented the bottom three stories of a ten story wall which were designed to characterize the typical construction practice of modern mid rise buildings on the west cost of USA. The primary variables of the test program were, shear demand, layout of the longitudinal reinforcement and continuity of vertical bars (spliced vs unspliced) in high plasticity zone where highest moment demand was expected. Test setup, overall dimensions and cross-section properties are shown in Figure 3.27.

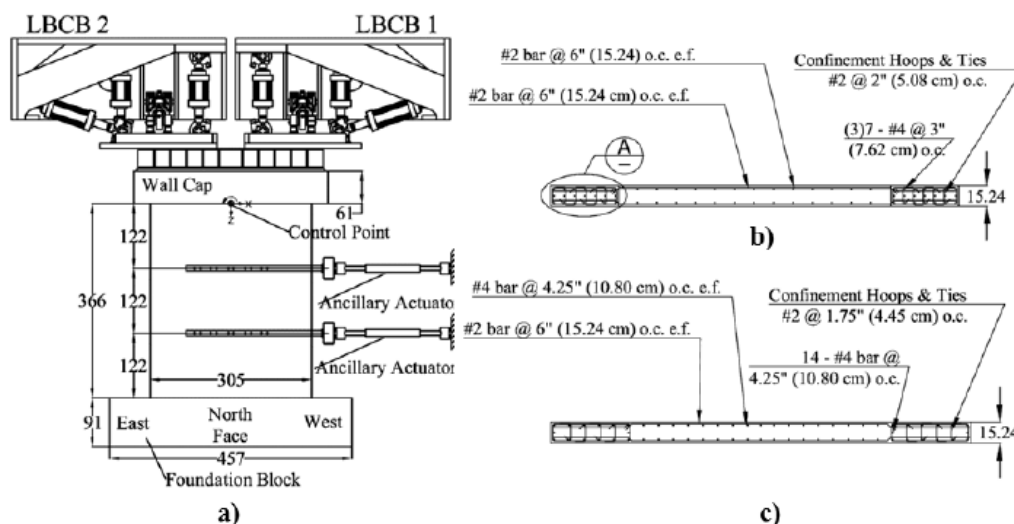


Figure 3.27 : UW/UIUC planar walls – a) Test setup and overall geometry; b) Cross-section for specimens PW1, PW2 & PW4; and c) Cross-section for specimen PW3.

All 4 rectangular test units were 12 ft. (3657 mm) tall, 10 ft. (3048 mm) long with a 6 in. (152mm) web thick. Axial loads applied to the specimen varied between $0.095 A_g f_c'$ and $0.13 A_g f_c'$. Design yield strength of $f_y = 60 \text{ ksi}$ (414 MPa) and design nominal compressive strength of $f_c' = 5.0 \text{ ksi}$ (34.5 MPa) were specified. Planar walls were labelled as PW1, PW2, PW3 and PW4.

3.5.1 Experimental results

3.5.1.1 PW1

PW1 was subjected to a loading that represented a standard triangular load distribution based on ASCE7-10 lateral force procedure. Horizontal cracking was first observed during a cycle to 0.1% drift whereas diagonal cracking commenced during the 0.15% cycle at inner edge of the boundary. Yielding of the vertical bars reported to be at a drift of 3%. Cover concrete spalling was first observed at 0.45% at the top of the spliced region. As the specimen pushed further, buckling of the longitudinal bars appeared at a drift of 1%. The drop in capacity was caused by the fracture of bars at the base of the wall. It is of importance to underline the fact that the tensile rupture failure of this bars at the base wall did not occur subsequent to buckling indicating a pure tensile rupture failure. Although rupture mechanism following buckling is one of the common flexural failure modes, pure tension failure is a rare one. The general responses of the wall are plotted in Figure 3.28.

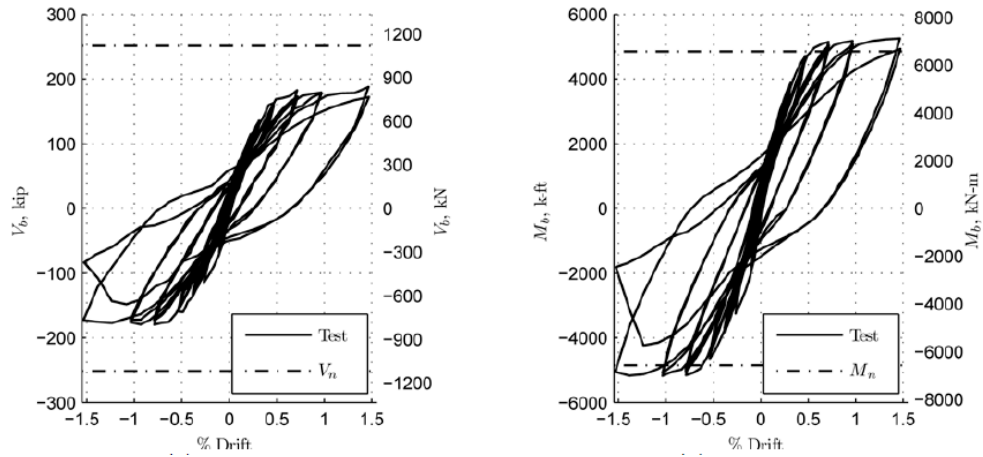


Figure 3.28 : UW/UIUC planar walls PW1 – a) Base Shear vs Drift b) Base Moment vs Drift.

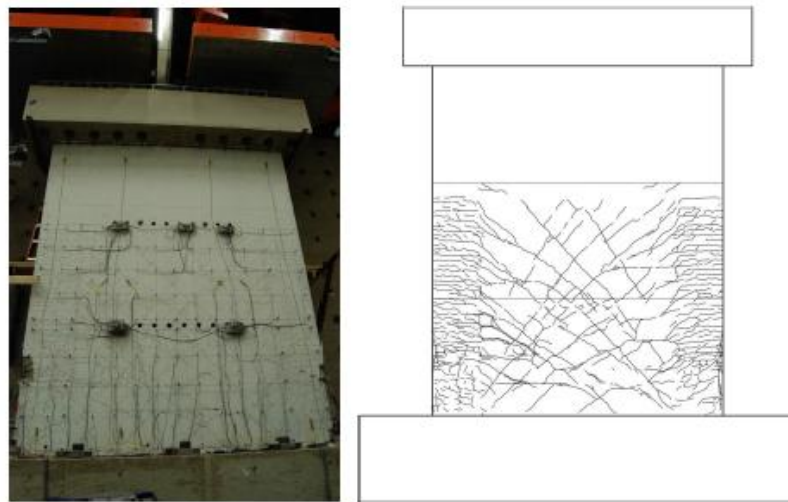


Figure 3.29 : UW/UIUC planar walls PW1 – Final Damage state and crack pattern.

3.5.1.2 PW2

PW2 was primarily designed to meet ACI code requirements with heavily reinforced boundaries. The moment and shear demands set to achieve an effective height of $0.5h$. Horizontal and diagonal cracking was first observed at a drift of 0.1%. yielding of the main vertical bars occurred at 0.45% drift. Cover concrete spalling commenced at the top of the spliced region at 0.53% drift and at the base of the wall at 0.75% drift. Buckling of the longitudinal bars were observed during a cycle up to 1%. The failure mode of the specimen which lead to a drop in lateral capacity was reported to be crushing of confined core at 1.5% drift. The Figure 3.31 demonstrates the test unit after the test. The general response plot was shown in Figure 3.30.

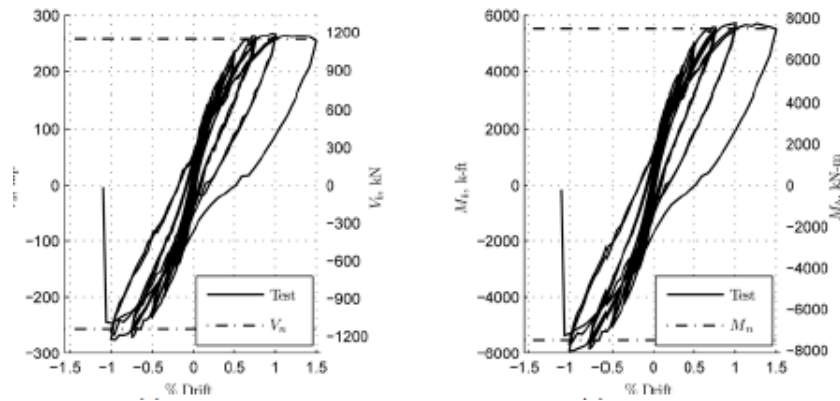


Figure 3.30 : UW/UIUC planar walls PW2 – a) Base Shear vs Drift ; b) Base Moment vs Drift.

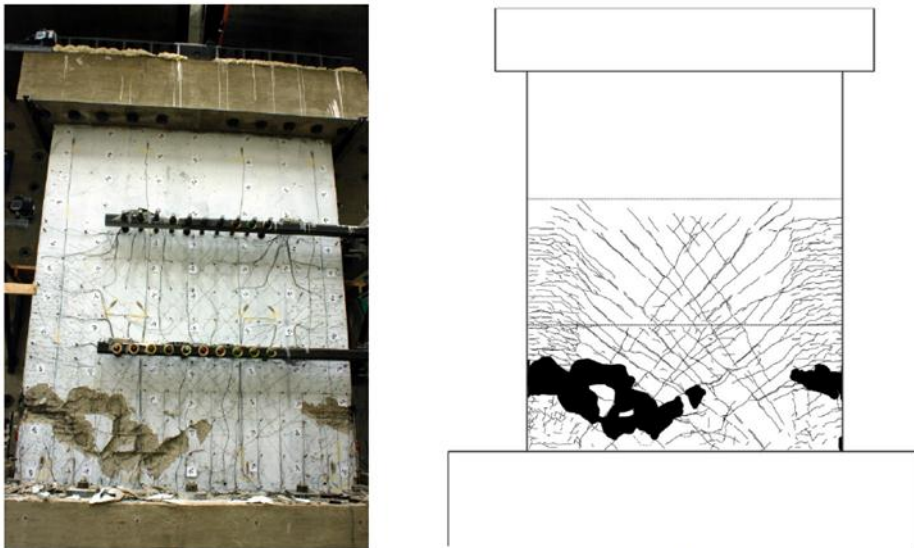


Figure 3.31 : UW/UIUC planar walls PW2 – Final Damage state and crack pattern.

3.5.1.3 PW3

PW3 differed from other specimens in terms of longitudinal reinforcement layout having a uniform distribution. The shear demand of PW3 was same as the one of PW2 and PW4 corresponding to an effective height of $0.5h$.

Horizontal and diagonal cracking initiated at 0.06% drift. Yielding of vertical bars was observed at 0.31% drift. Spalling of cover concrete was observed at the top of spliced region at a drift of 0.35%. Bar buckling was observed at 0.75% drift. Failure occurred due to extensive bar buckling and core crushing in the east boundary element above the splice at 1.28% drift. The final damage state of the wall can be seen in Figure 3.35

The load-deformation hysteresis was shown in Figure 3.32.

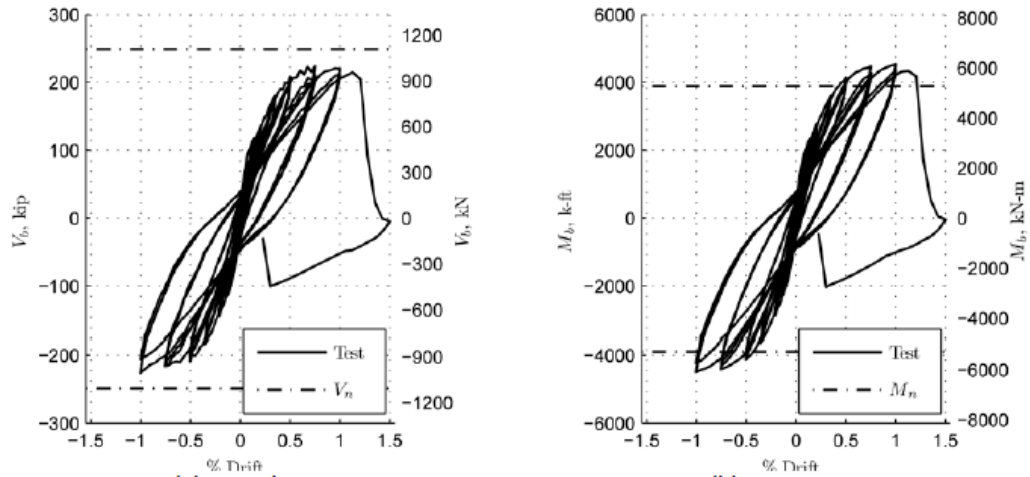


Figure 3.32 : UW/UIUC planar walls PW3 – a) Base Shear vs Drift b) Base Moment vs Drift.

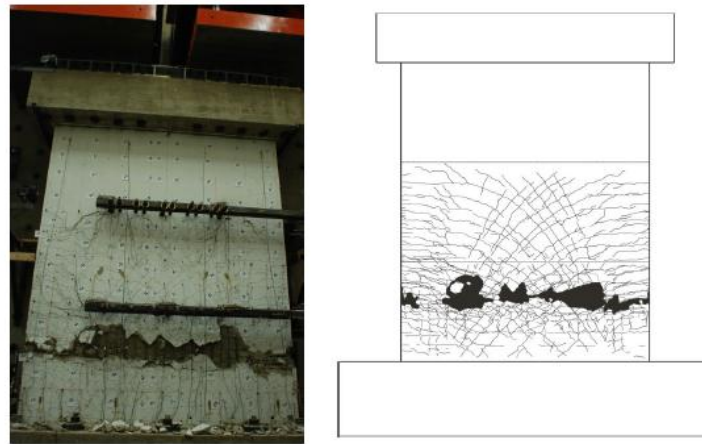


Figure 3.33 : UW/UIUC planar walls PW3 – Final Damage state and crack pattern.

3.5.1.4 PW4

PW4 was the only one with a continuous longitudinal reinforcement without any splice. The other features of the specimen were identical to the those of PW2 & PW3. Key damage states were as follows; horizontal cracking initiated at 0.06% drift and diagonal cracking at 0.07% drift. Yielding of outermost bars was indicated by strain gauges at 0.30% drift. Cover spalling was observed at the toe of the wall during the cycle to 0.50% drift. Bar buckling was revealed at 0.75% drift. Failure occurred due to extensive bar buckling and core crushing in the east boundary at the toe of the wall during the cycle up to 1.0% drift. Figure 3.35 shows the final damage state of the specimen and the load-deformation plots are shown in Figure 3.34.

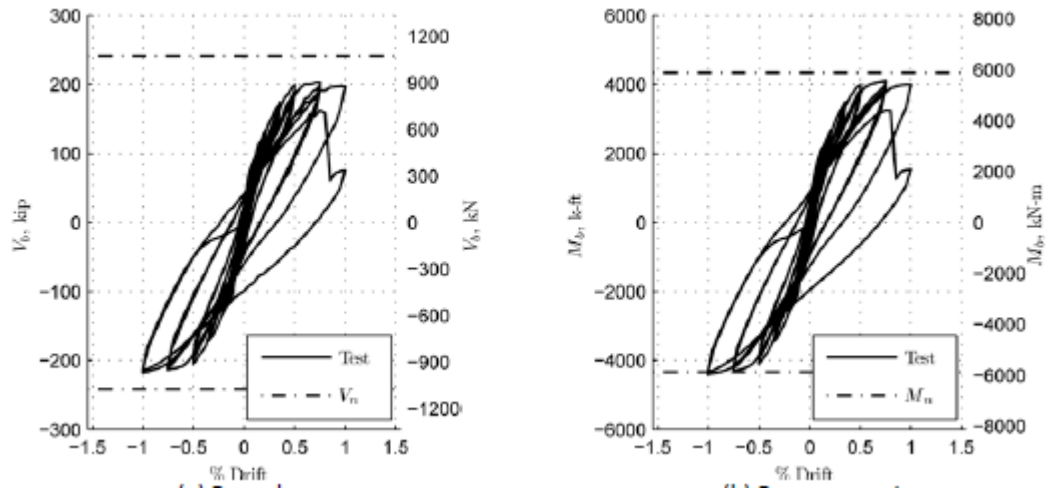


Figure 3.34 : UW/UIUC planar walls PW4 – a) Base Shear vs Drift ; b) Base Moment vs Drift.

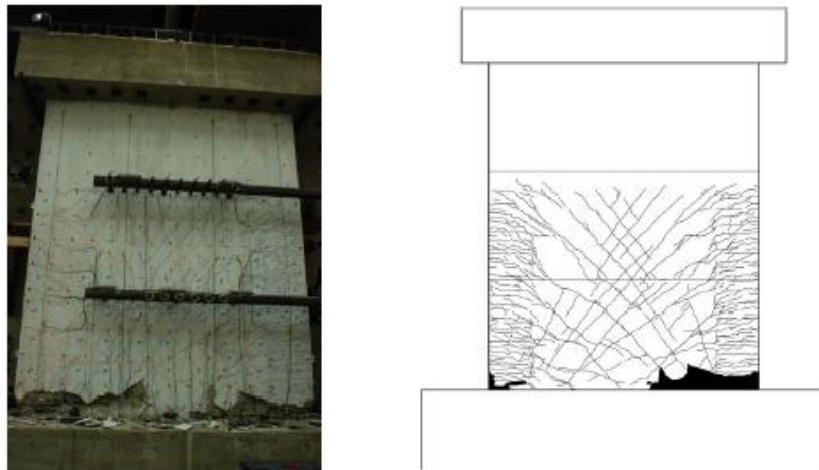


Figure 3.35 : UW/UIUC planar walls PW4 – Final Damage state and crack pattern.

3.6 Kono et al. (2011, 2012)

The authors conducted a series of tests on 9 rectangular and barbell-shaped RC walls which were subjected to quasi-static cyclic loading. The main objective of the study is to investigate the ultimate drift capacity and progress of the damage. The primary variables of the test program were; cross-section shape, reinforcement layout and the level of axial load. Test setup, overall dimensions and cross-section properties are shown in Figure 3.36, Figure 3.37 and Figure 3.38.

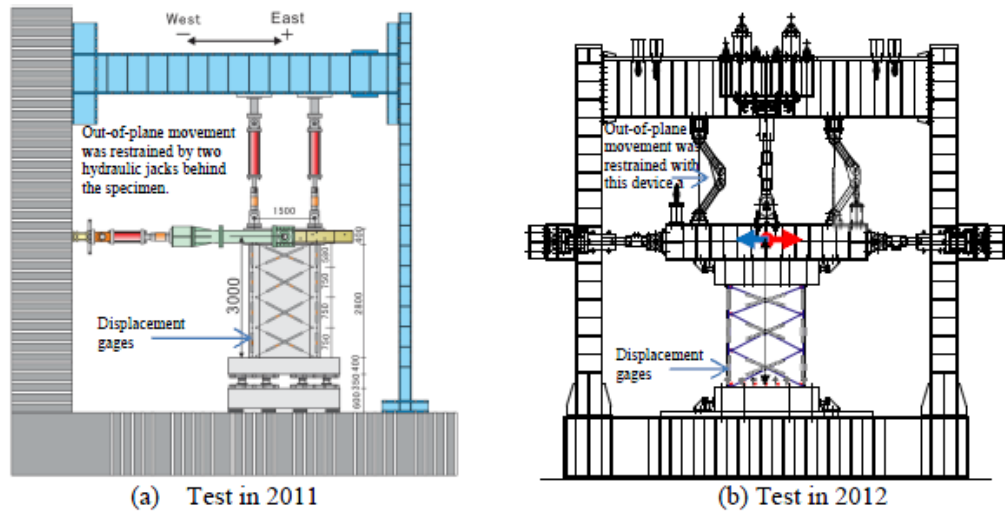


Figure 3.36 : Kono et al. (2014) – Test setups.

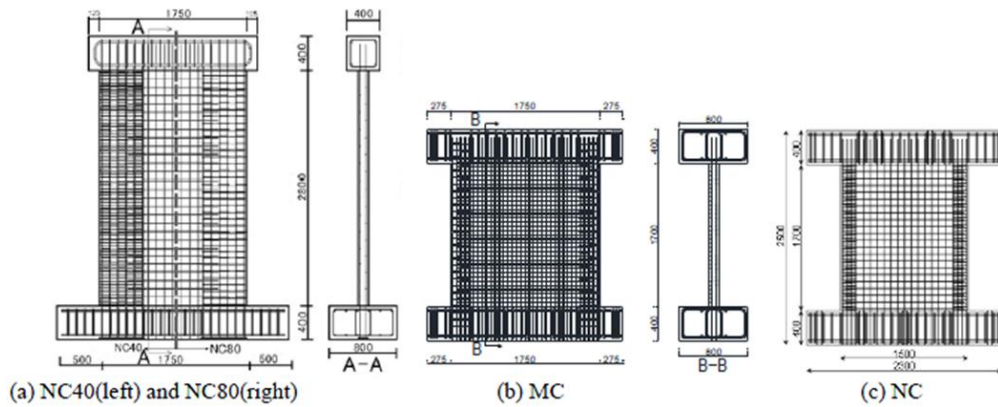


Figure 3.37 : Kono et al. (2014) – Overall dimensions of NC40, NC80, MC and NC.

7 of the specimens were 1750 mm long and 2800 mm tall. The web thickness of NC80 and NC40 was 128 mm while NC, SC and HN having a 1200 mm thick web. Barbell-shaped BC40 and BC80 specimens had boundaries with dimensions of 250 mm *250 mm and a web with 80 mm thickness. On day of test material properties were as follows; yield strength for longitudinal steel reinforcing varied between 348 MPa and 385 MPa and compressive concrete strength ranged between 27.5 MPa and 59.5 MPa. The axial load ratio ranged between $0.03 A_g f_c'$ and $0.11 A_g f_c'$.

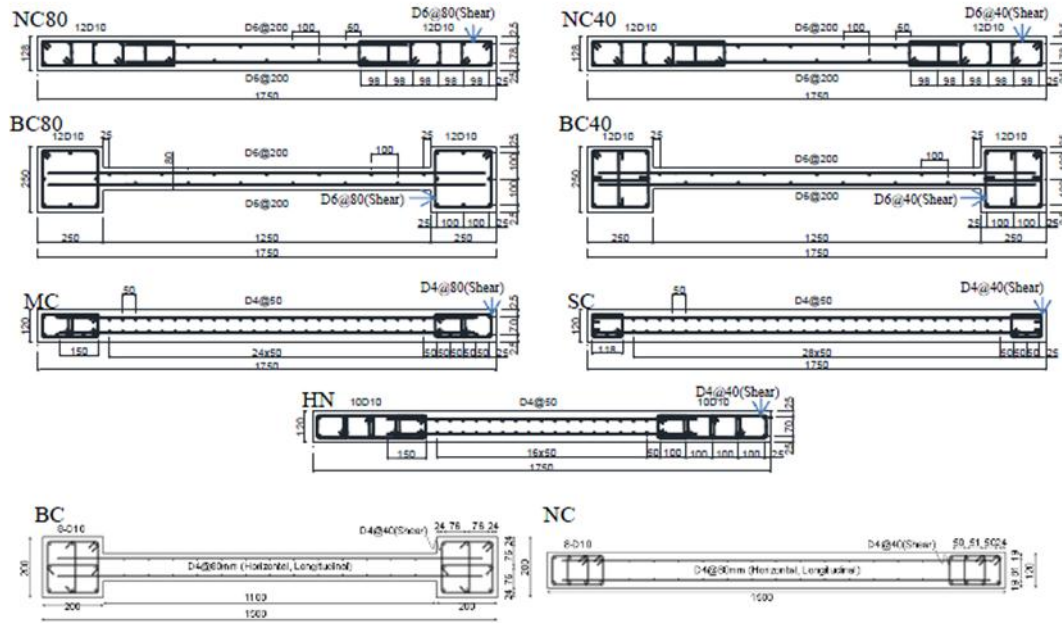


Figure 3.38 : Kono et al. (2014) – Cross section geometries of test specimen.

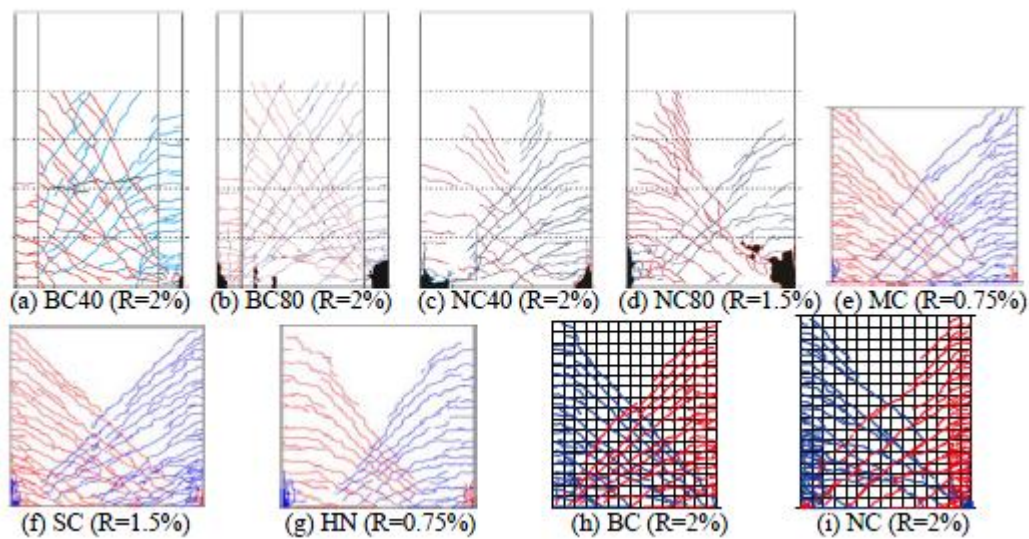


Figure 3.39 : Kono et al. (2014) – Damage state at various drift ratios.

Damage progression of walls as follows, the diagonal and horizontal cracks propagated over the wall panel until the ultimate load (Figure 3.39). Spalling of cover concrete for the specimens were generally seen between at the drifts of 0.5% and 1%. Buckling was revealed after termination of the tests. Thus, from the readings of strain gauges, buckling was considered to have occurred prior to reaching to the peak strength. Buckling of longitudinal reinforcement generally did not play a major role in the strength loss. However, buckling of bars was likely to impose larger transverse strain demands on hoops/cross-ties leading to fracture of hoops and/or crushing of core

concrete. Failure of the specimens was due to crushing of the confined region at various drift levels. The crack patterns at the end of the test and the load-drift ratio relation are shown in Figure 3.40.

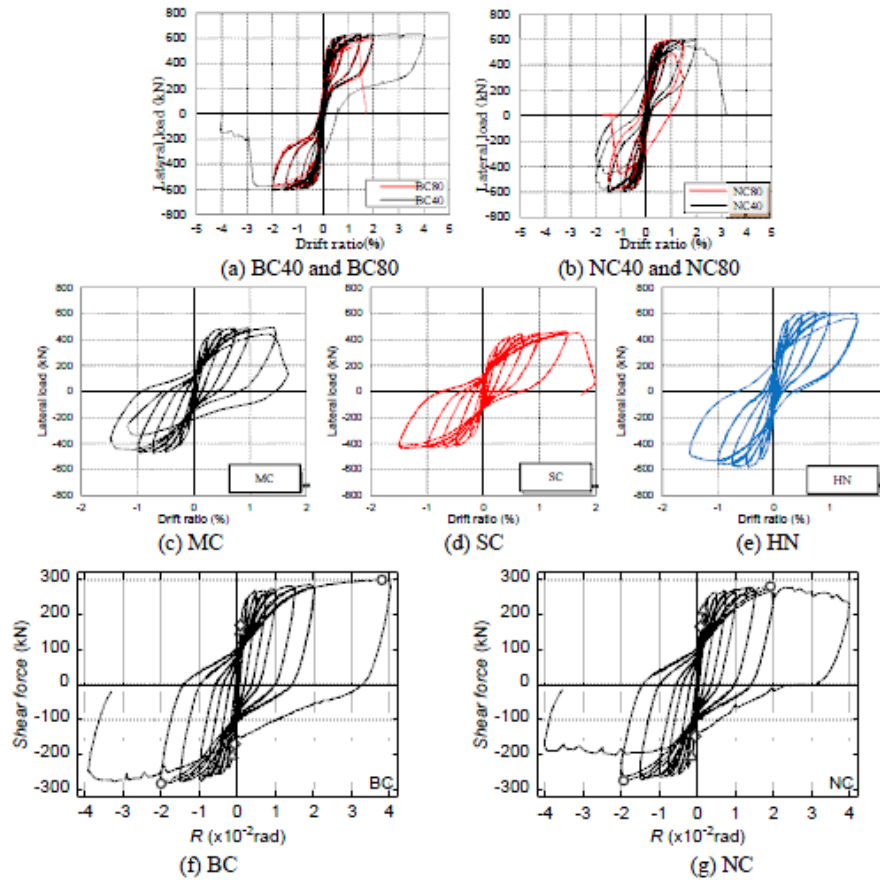


Figure 3.40 : Kono et al. (2014) – Load-deflection curves.

3.7 Hines et al. (2002)

Hines et al. (2002) conducted a test program for hollow rectangular reinforced concrete bridge piers which was completed in three phases. The first phase was set to investigate the flexural behaviour of shear walls with highly confined boundary elements which were subjected to cyclic loading. The test units were not designed to be representative of a specific bridge pier, but instead intended to reflect the general characteristics of Bay Area bridge piers. The experimental program primarily focused on the variables including column length, quantity of transverse reinforcement and wall thickness. In the first phase, the test units had an aspect ratio of $M/VD=4$. The specimens had identical boundary elements whereas they differed in the transverse reinforcement of web panel. The orthogonal boundary elements were set at 12in. (305mm) inscribed diameter and spaced 24 in. (610 mm) apart connected by a 6 in. (152 mm) thick web

panel (Figure 3.41). The first phase test units namely 1A and 1B had a section depth of 1219 mm and a height of 4877 mm. The second phase test units (2A-2B) had identical cross-section geometry and reinforcement layout to those of the first phase test units but the height for 2A-2B is the half as much as the height for 1A-1B which made them have an aspect ratio of $M/VD=2$ instead of $M/VD=4$. The test day compressive strength of concrete ranged between 5.31 ksi (36.6 MPa) and 6.21 ksi (42.8 MPa). Yield strength of the longitudinal bars for the first and the second phase test units were 67 ksi (462 MPa) and 66 ksi (455 MPa) respectively. The axial load levels were intended to be set at $0.10 A_g f_c'$ but the applied axial loads varied between $0.083 A_g f_c'$ and $0.11 A_g f_c'$. Test setup and overall dimensions are shown in Figure 3.42 and Figure 3.43.

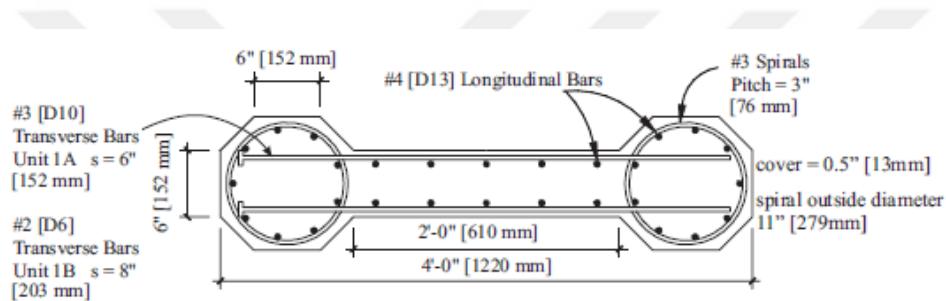


Figure 3.41 : Hines et al. (2002) – Section geometry and reinforcement layout of 1A, 1B, 2A and 2B.

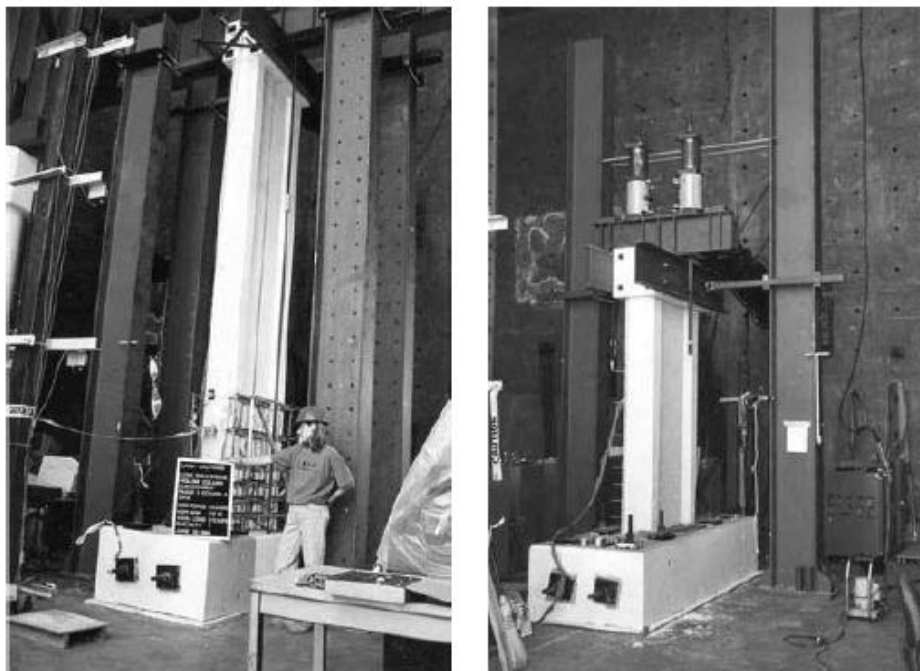


Figure 3.42 : Hines et al. (2002) – Test setup of Phase 1 and Phase 2 Units.

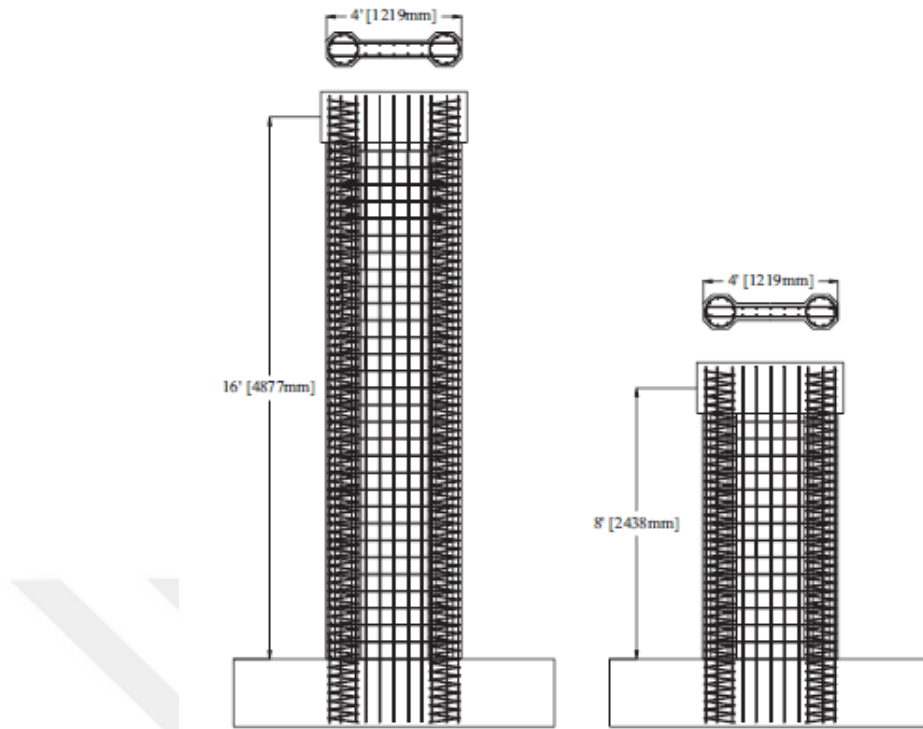


Figure 3.43 : Hines et al. (2002) – Overall dimensions of Phase 1 and Phase 2 Units.

3.7.1 Test observations

3.7.1.1 Unit 1A

Test unit 1A exhibited a flexural behaviour in general. Flexural cracks spread over the halfway up the boundary element height. The damage progression was as follows;

Horizontal cracks initiated at a load of $F=31.9$ kips (142 kN) which approximately corresponds to $\frac{3}{4} F_y$. Spalling of cover concrete was first observed during the cycle $F=66.0$ kips (293.7 kN). After the third excursions to $\mu_\Delta = 3.0$, chunks had fallen off.

Buckling of longitudinal bars was first observed during the first cycle of $\mu_\Delta = 6$ and fractured during the third cycle of $\mu_\Delta = 6$ which caused severe strength degradation. All the bars fractured during the cycle up to $\mu_\Delta = 8$ leading to the full degradation of the tension boundary elements (Figure 3.44). The load-deformation loops of hysteretic behaviour are shown in Figure 3.45.



Figure 3.44 : Hines et al. (2002) – Test unit 1A fractured longitudinal bars at the end of the test.

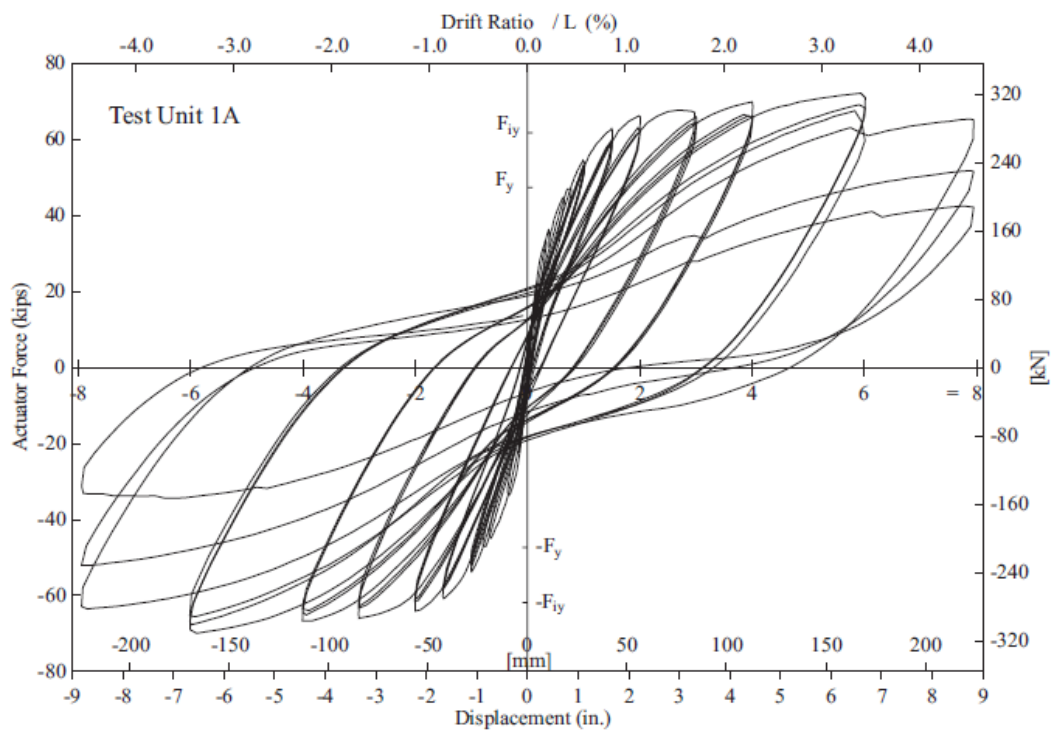


Figure 3.45 : Hines et al. (2002) – Test unit 1A Load – deformation hysteresis.

3.7.1.2 Unit 1B

The performance of test Unit 1B was governed by flexural behaviour. Progression of damage is as follows; horizontal cracking was initiated at a load of $F=33.5$ kips (149.1 kN). Initiation of spalling was observed at a displacement ductility level of $\mu_{\Delta}=3$. Onset of buckling was first observed during the third cycle to $\mu_{\Delta}=6$. Three of the vertical bar fractured while the specimen pulled in the negative direction up to $\mu_{\Delta}=-2$. Several more bars had experienced the tensile rupture failure when loading cycle reached $\mu_{\Delta}=8$ (Figure 3.47). Load-deformation relation is shown in Figure 3.46

“The hysteresis loops showed that once a bar fractured and the force dropped, the column gradually regained its strength with increased deformation by shifting the tensile demand to bars further inside the column” (Hines et al. 2002, p.112).

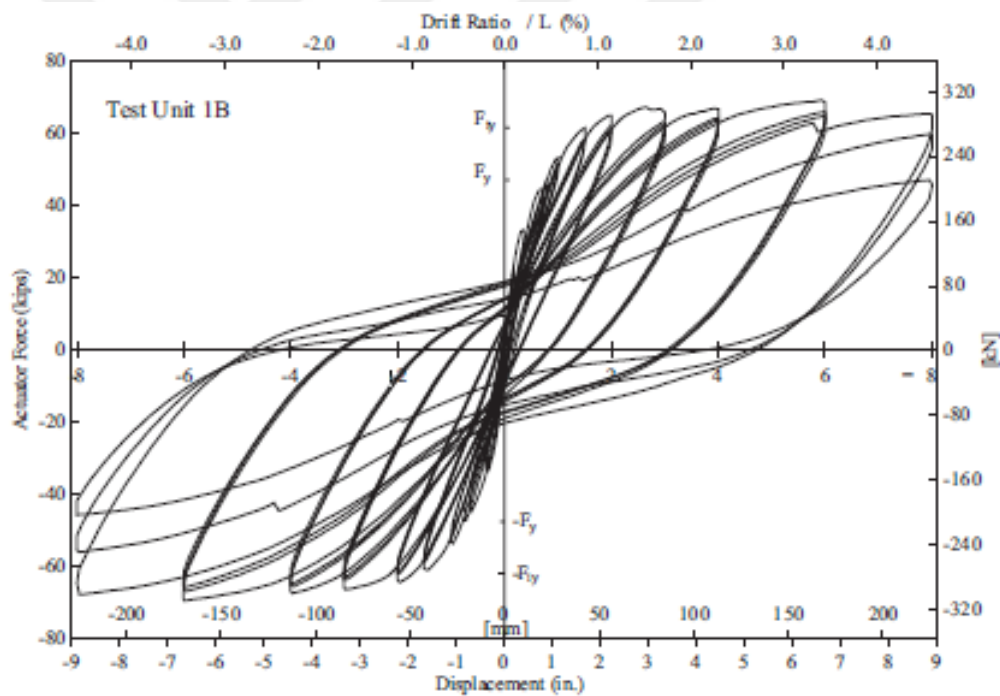


Figure 3.46 : Hines et al. (2002) – Test unit 1B Load – deformation hysteresis.

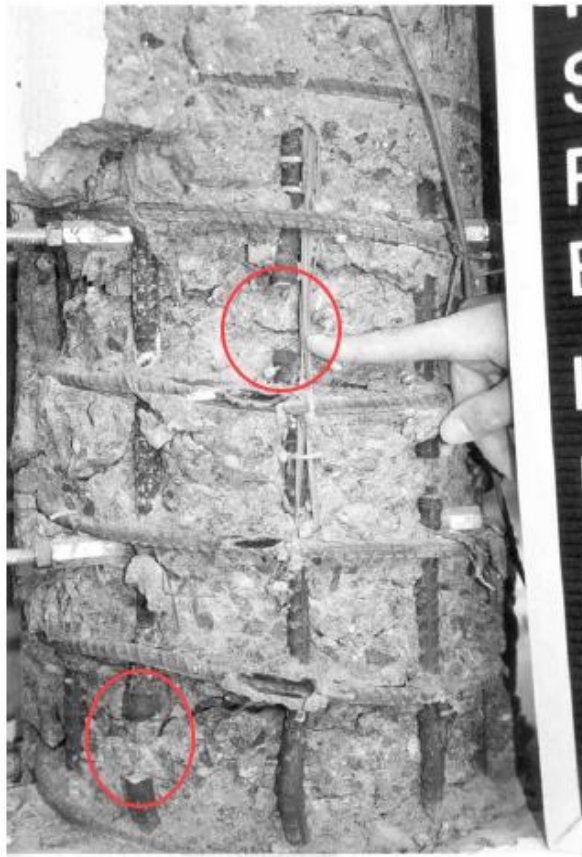


Figure 3.47 : Hines et al. (2002) — Test unit 1B fractured longitudinal bars at the end of the test

4. ANALYTICAL MODELING OF DATASET SHEAR WALLS

There are several modeling approaches that have been employed with the purpose that is to simulate the behavior of reinforced concrete walls. Some of which requires more computational effort in terms of modeling but captures the response elaborately, while some can represent the basic characteristics of the behavior with a few fundamental assumptions. The simplest and the most basic approach is the lumped plasticity models which assumes that the plasticity zone concentrates in one flexural plastic hinge near the largest moment demand. The plastic curvature, in this model, are assumed to be constant along with the plasticity zone and the elastic curvature considered exhibiting a linear triangular distribution over the height of a cantilever wall height. One should note that this curvature profiles assumed by lumped plasticity models are only an approximation of the physical behavior however owing to simplicity, the PHA method is widely favored by practicing engineers and researchers. The basic procedure of predicting a displacement capacity through the PHA method based on local limits is as follows; initially a moment-curvature analysis is needed to be conducted and then based on adopted strain limit values (ϵ_{su} , ϵ_{cu}) and plastic hinge length estimation l_p , global response of a RC shear wall can be calculated. The accuracy of the above mentioned procedure relies on three legs;

1. Adopting the right failure criterion. For instance, if one adopts a tension side strain limit " ϵ_{su} " to assess the drift capacity of a wall, of which failure mechanism is crushing of confined core, it would clearly yield unreasonable results overestimating the actual capacity. In the following section, an expression will be proposed to distinguish the flexural failure modes from each other so that the ultimate strain limit would be compatible with failure mode which in return will minimize the mistakes induced by choosing the wrong side strain limit. Specific samples of the application of the PHA method to dataset shear walls will be covered as to indicate how unreliable the assessment of displacement capacity could become especially when the wrong side failure criterion is picked.

2. Reliability of strain limits proposed in the literature. The validity of strain limits for shear walls is limited to some extent on the ground that strain limits are mostly calibrated on sections that are subjected to pure axial compression or tension effects without accounting for axial load-moment interaction (i.e strain gradient). The results of the PHA method in comparison with experimental results will be given to evaluate the strain limits reliability.
3. Determining the spread of plasticity prior to application of plastic hinge modeling is of crucial since a larger estimation of plasticity zone would in return give unconservative predictions or a smaller plastic hinge length " l_p " would underestimate the drift capacity of the wall. The distinction between the results which are obtained via employing different plastic hinge lengths " l_p " will be shown.

4.1 Assembling of Dataset Shear Walls

A dataset of 49 shear wall specimens that previously tested has been assembled to evaluate the modeling approach discussed above. The test units that satisfy the distinct criterions have been picked from the literature. The criterions are as follows;

- Planar (Rectangular shaped) wall specimen subjected to in plane flexure, shear and axial loading. There has been an ongoing discussion for T,U-shaped walls whether the plane section assumption for this type of walls is valid or not. To avoid complexity, those section types are excluded.
- Test units of which behavior were mainly governed by flexure and end up with flexural type failure mode has been selected. In this study, flexural failure mechanisms refer to crushing of core, rupture of previously buckled bars and tensile rupture of bars due to low-cycle fatigue.
- Performing a numerical model requires to have some basic data of specimens. Cross-section geometry, overall dimensions, reinforcement layout and material properties of reinforced concrete member i.e concrete compressive strength or stress-strain of reinforcing steel were provided.
- Global load-displacement responses of specimens were provided as to be able to make a comparison with simulation results.

Overall specifications of test specimen of dataset

- Dataset comprised of 49 walls from 11 different research groups. Axial load ratios for test units varied between $0.003 A_g f_c'$ and $0.202 A_g f_c'$. One of the key evaluation criterions, section depth to length of wall " c/l_w " ranged between 0.05 and 0.33.
- Aspect ratios L_v/l_w differed between 1.33 and 4. Normalized shear stress values $V/A_g \sqrt{f_c'}$ varied between 0.07 and 0.47.
- Test specimens in general comply code requirements in terms of boundary elements. Single perimeter hoop with supplemental cross ties, overlapping hoops or piecewise hoops were employed to confine the boundary zone. One specimen does not have stabilizing transverse bars WSH4 (Dazio et al, 2009). For the rest of the dataset walls, volumetric confining bars ratio " ρ_s " was between 0.0021 (lowest) for B1 (Oesterle et al, 1979) and 0.0411 (highest) for WSH5 (Segura 2018). $\frac{A_{sh} * f_y}{s * b_c * f_c'}$ ratio varied between 0.012 and 0.28.
- s/d_{bl} ratio for specimens varied between "2.68" for R2 (Oesterle et al, 1979) and "15.38" for WR20 (Oh et al. 2002). s/d_{bl} plays a major role in prevention of local buckling of longitudinal bars.
- Concrete compressive strength was $27.50 < f_c' < 86.08 \text{ MPa}$
- Ratio tensile-to-yield strength of boundary reinforcement f_y/f_u was 1.13 at min and 1.68 at max.
- Wall web thickness t_w was larger than 100 mm for all specimens.
- 30 shear wall in the database experienced compressive crushing of core area accompanied with buckling of vertical bars, which will be referred as "CB", 15 of dataset walls failed due to rupture of previously buckled bars "BR" and only 3 of the specimens have failure mode of pure tensile rupture of longitudinal bars "R0". It is noteworthy to state that this type of failure mechanism "R0" is not truly common in the field and mostly observed for the laboratory test walls. (Figure 4.1)

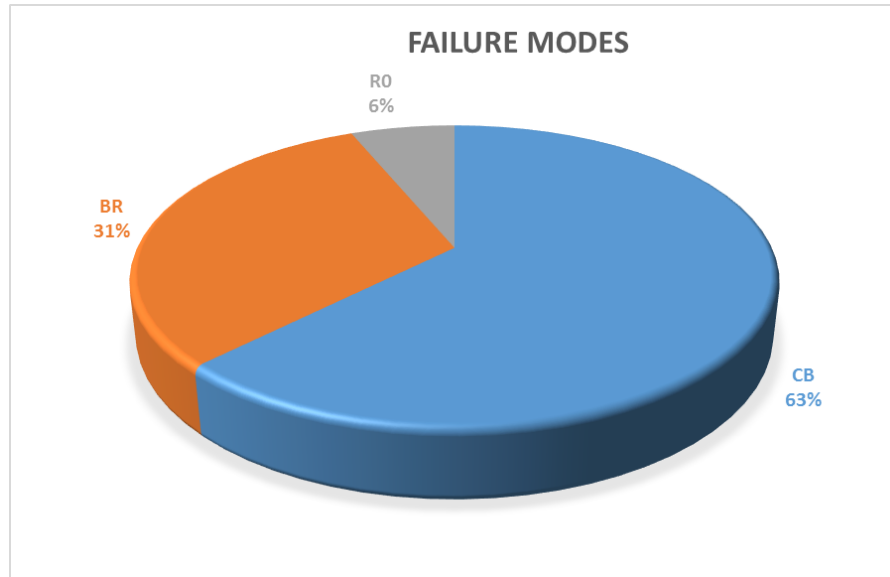


Figure 4.1 : Failure modes of dataset shear walls.

A fiber section model was developed to conduct the moment-curvature analysis of dataset walls via utilizing “Opensees” platform. “Opensees” allows the user to fully define the material properties of concrete and reinforcing steel. Given the fact that it was only a 2D cross-sectional simulation of specimen, the modest material models, which does not account for cyclic load effects or degradation, were used in modeling. For reinforcing steel “steel02” and for concrete “concrete01” was chosen. To reflect the impact of confinement on both strength and ductility, the constitutive model proposed by Mander et al., (1988) was used. This model simply considers the effect of confinement.

4.2 Anticipation of Flexural Failure Modes

Failure modes from flexure to out of plane instability summarized in Figure 4.2. In this study, the main focus is on flexural type failure modes which basically occur after flexural yielding of reinforcing bars before any shear mechanism take places. In other words, the nominal shear strength capacity and the web-crushing capacity of the member is expected to be larger than the nominal moment capacity (Figure 4.3).

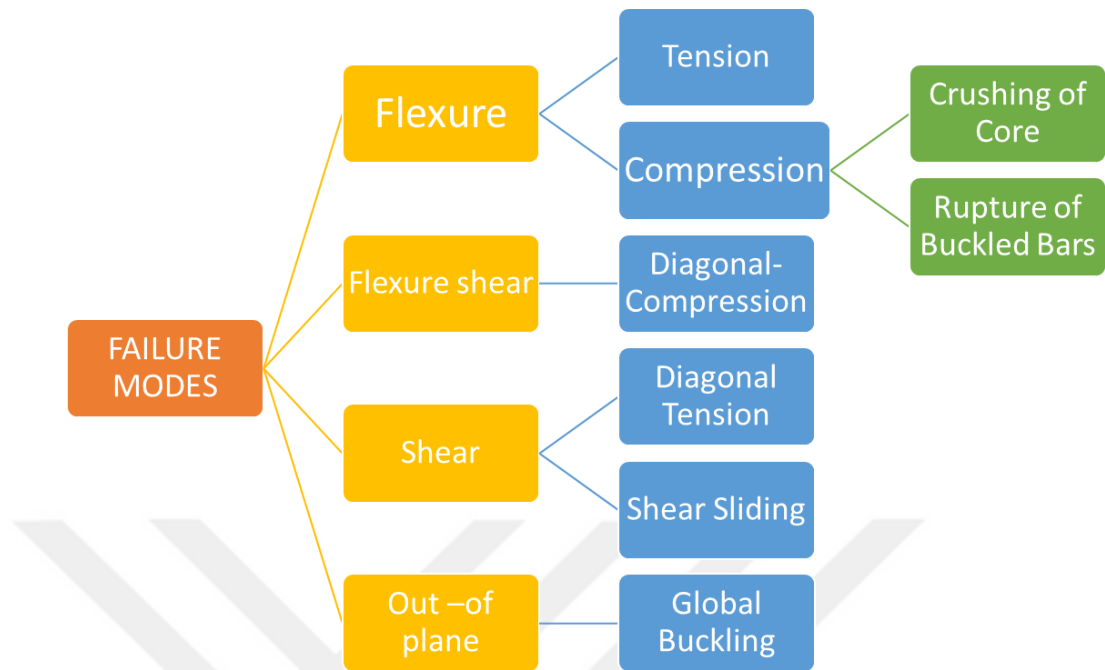


Figure 4.2 : Failure modes.

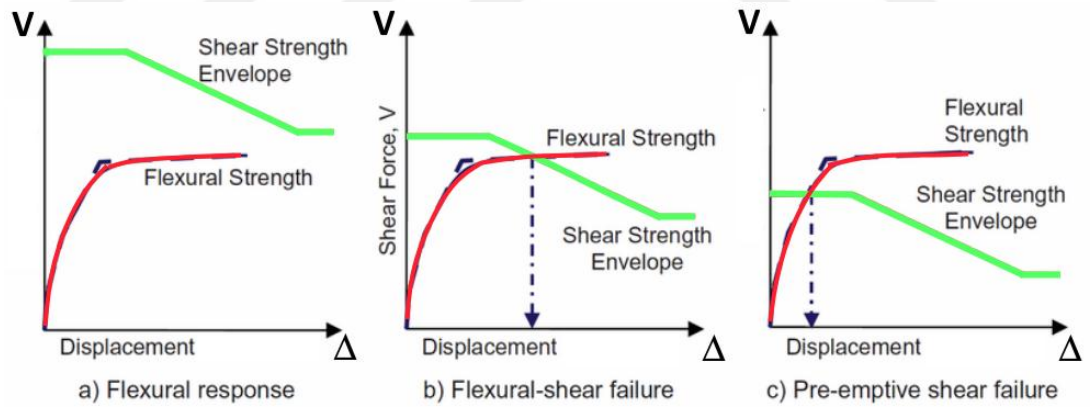


Figure 4.3 : Force-displacement relation for failure modes.

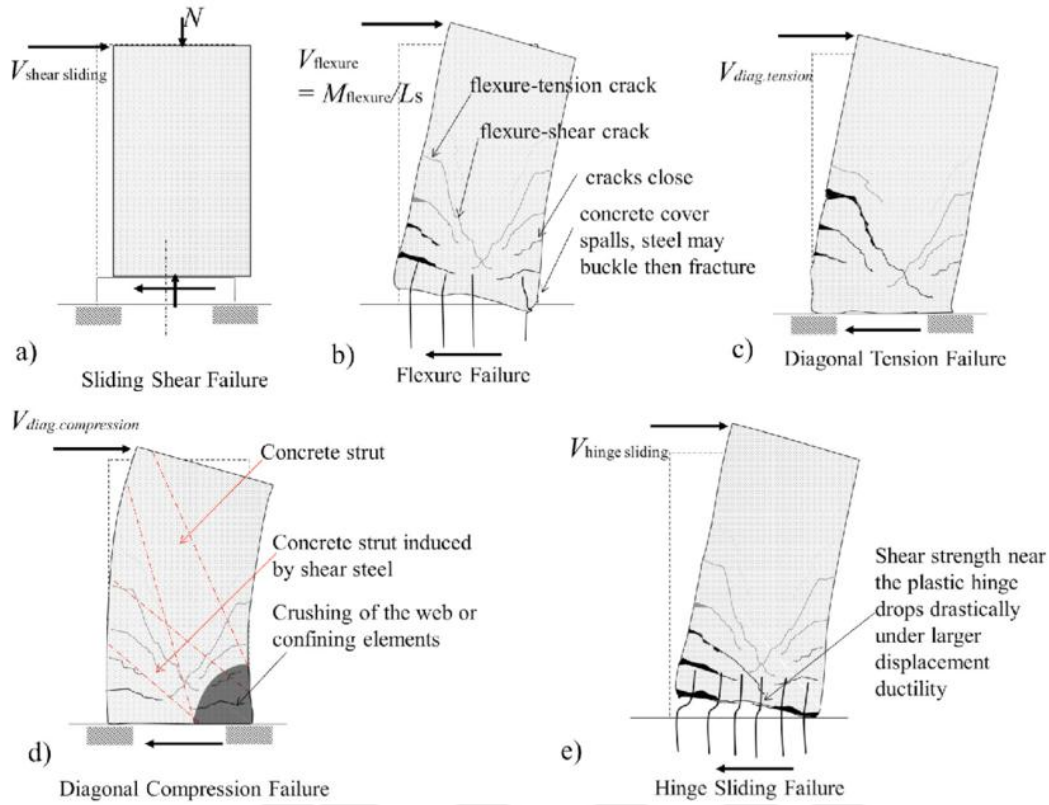


Figure 4.4 : Typical shear wall failure mechanisms (Tang and Su, 2014).

When it comes to adopting a failure criterion for plastic hinge modeling approach with assessment purposes, it is highly crucial to anticipate which failure mode to take place. It is relatively easier to distinguish shear critic failures from flexural critic failures by following one of the procedures suggested in the literature. However, it is also important to be able to differentiate whether crushing of concrete or rupture of previously buckled bars will cause the strength degradation since adopting a tension side criterion for compression side would not estimate accurately or the other way around. The previous research demonstrates that upper-bound limit of fracture of previously buckled bars is primarily a function of s/d_{bl} and ϵ_{su} while compressive failure is mostly related to ϵ_{cu} . From the results of the moment-curvature analysis of the dataset shear walls of which spacing of transverse bars " s " is smaller than $8d_{bl}$, an expression to distinguish failure modes of flexure has been developed;

$$m = \frac{c}{b} * \frac{\epsilon_{su}}{0.1} * \frac{14 - \frac{4}{3} \frac{s}{d_{bl}}}{100} * \frac{c}{l_w} * \frac{1}{\epsilon_{cu}} \quad 4.1$$

Where c is the section depth at nominal moment point corresponding to the extreme fiber compressive strain of 0.004 or extreme fiber tensile strain of 0.015 whichever

occurs first, b is the boundary zone thickness. ε_{su} is the strain of reinforcing steel at maximum stress (i.e. tensile strain capacity), ε_{cu} is the confined concrete strain capacity.

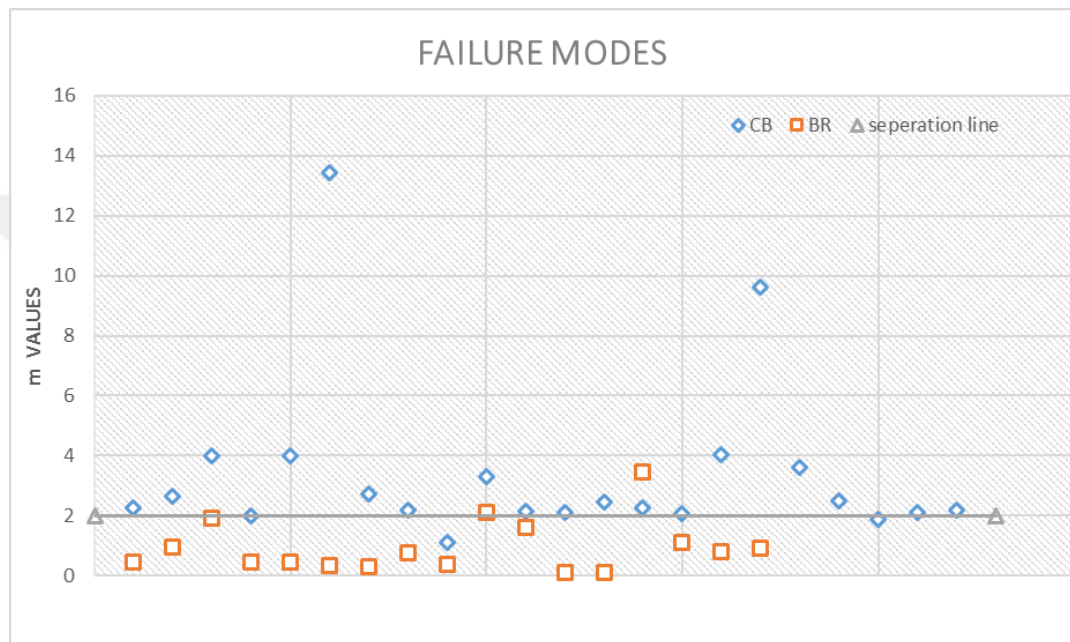


Figure 4.5 demonstrates that the larger the m values get, the more likely crushing of core concrete occurs. The mean of m values for BR was 1.15 whereas the mean values for CB was 3.66. It is also noteworthy to conclude from the results given above, for the specimens of which m value is closer to separation line, the displacement capacity prediction with tension side strain limits would yield slightly more reasonably.

The results of the application of the plastic hinge analysis method to WSH2 (Dazio et al. 2009) is as follows; The m value for the specimen is 0.49 which means that strength degradation is due to rupture of buckled bar. Hypothetically if one adopts compressive concrete strain limit ε_{cu} in order to predict the displacement capacity of

this test unit via plastic hinge method, the displacement capacity would be found as 121.56 mm which overestimates 1.93 times of the actual value of displacement (62 mm). This is definitely a huge gap for an assessment procedure. Note 1 ; To avoid the discussions which would stem from the choice of plastic hinge length estimation, the calculations were done with the given plastic hinge length in the experimental report. Note 2; Shear contribution to overall displacement was accounted by the expression of Beyer et al. (2011).

The other application of plastic hinge method with the improper ultimate limit is to HN (Kono et al. 2012). The m value for specimen is 13.41 which means that the failure mode of the specimen is crushing of concrete. If tension side strain limit ϵ_{su} is adopted with the aim of predicting the displacement capacity of this test unit via plastic hinge method, the displacement capacity would be found as 91 mm which overestimates 2.53 times of the actual value of displacement (36 mm).

What happens if a specimen of which m value is closer to “2” is to be applied plastic hinge modeling with the wrong side ultimate limit. The next example will try to demonstrate the results. WSH6 has a m value of 2.05, which failed due to crushing of core. The prediction with tension side limit would overestimate the capacity 18% larger whereas the compression side would yield the ultimate displacement 12% larger than the actual one. As a result, there is no profound distinction in prediction of displacement capacity with regard to the final failure mode when the m value gets closer to “2”.

There is one more example for which the failure index presented in this study depending on m value predicts the failure mode wrongly. The test unit MC (Kono et al. 2012) has a m value of 1.98 which indicates a BR failure, however, the actual physical failure is CB. Plastic hinge modeling is applied to this specimen adopting BR failure criterion, which estimates the ultimate displacement as 50.96 mm while the plastic hinge method with compression side failure limit, it yields the displacement capacity as 53.02 mm . Although the failure criterion is misfigured, the estimated results can be definitely regarded as tolerable in terms assessment purposes.

The procedure for estimating the experimental displacement capacity of above mentioned examples with the PHA method is based on the following assumptions, (the validity of these assumptions are limited to the above examples, different

assumptions are utilized through the application of the PHA method through this thesis)

- The plastic hinge lengths are taken as they were given in the experimental reports in order to avoid any error which would be attributed to the choice of the plastic hinge length estimations.
- The tension side strain limit ϵ_{su} is calculated according to the proposal of Restrepo (2006). Given in Section 2.5.
- The ultimate strain limit associated with compressive crushing of core concrete is adopted as Mander et al. (1988)'s equation.
- The shear contribution to total lateral deformation is accounted by the proposal of Beyer et al. (2011).

4.3 The Investigation of the Reliability of Ultimate Strain Limits

The local scale limitation values play a major role in a standard procedure for determining global capacities by utilizing the PHA method. The importance of adopting the strain limit properly to the failure mode has been explained in previous section. On the other hand, the accuracy of strain limit value itself is as much significant. There are, of course, a few proposals in literature for both the extreme tensile fiber and the extreme compressive fiber however researchers have argued the reliability of these strain limits especially compressive strain limits for shear walls under axial load and flexure. Hines (2002) criticized the approach by Mander et al. (1988) since the effect of longitudinal bar buckling was not considered in that expression. Hines (2002) stated that buckling of the vertical bars may lead to local yielding and subsequent fracture of the hoops. Hines (2002) argued that if the fracture of hoops primarily caused by bar buckling rather than expansion of core concrete under compression the energy balance approach by Mander et al. (1988) is erroneous.

The PHA method with compressive strain limits is performed on some of the test specimens which failed for crushing of core concrete, in order to demonstrate how unreliable the assessment of global displacements could get when the ultimate strains are estimated incorrectly by the equation of Mander et al. (1988). For the specimen RW2 (Thomsen and Wallace, 1995), the ultimate compressive strain capacity by equation of Mander et al. (1988) equals to $\epsilon_{cu} = 0.0132$. If this local scale value is to

be transformed into a global scale value by the PHA method, the displacement capacity would be found as 136 mm which is 1.59 times larger than the real lateral displacement (86 mm). This, of course, may be attributed to the quality of plastic hinge estimation and one can argue that the improper plastic hinge length estimation would lead to this overestimation. However, when the displacement capacity is recomputed with the lowest plastic hinge length estimation (equation 2.24) for this specimen (which approximately equals to $0.24l_w$), the displacement capacity is still overestimated by 20%. Above result simply legitimate the criticism levelled at the compressive strain limits applicability to the shear walls. Although there are some reservations within the engineering community regarding the mechanical principles behind the equation (2.6), it still remains one of the widely used expressions for estimating the strain capacity of confined concrete sections.

One other striking sample on the reliability issue of compressive strain limit is the results of section analysis of test specimen WP7 (Segura 2017). The failure mode of the specimen is reported to be abrupt crushing of core which means the compression side limitations are critical and do govern the drift capacity. The strain capacity of concrete is $\epsilon_{cu} = 0.0254$ by Mander et al. (1988) which corresponds to a reinforcing steel strain of $\epsilon_s = 0.156$ (clearly beyond ϵ_{su}) at moment curvature analysis. The section analysis results may lead one to think that the specimen would have failed due to bar rupture failure but it actually did not. The only logical explanation for this situation is that the compressive strain capacity is estimated considerably larger than its actual value.

There is one more example which demonstrate the overestimation of compressive strain capacity. The test unit W4 (Chen 2005) had experienced concrete crushing failure. The ultimate curvature measured experimentally as $\phi_{base} = 4.39e - 05/\text{mm}$. At this very curvature, the concrete and steel strains are obtained as 0.00762 and 0.04032 respectively from moment-curvature analysis whereas the estimation of Mander et al. (1988) for this specimen is 0.0216 which is clearly greater than actual strains.

The conflict between actual compressive strains and estimated ones discussed in above section may be attributed to the uncertainties which are listed as follows;

- Uncertainty of ultimate tensile strain capacity of confining bars. The stress-strain relation of reinforcing bars with smaller diameter (generally preferred as to confine the boundary zone of test units) are not dependable as those of larger diameter bars.
- Uncertainty of the distribution of transverse strain demands to individual hoops and cross-ties. For instance overlapping hoops are known to have evenly distributed transverse strain demands to each legs which in return increase the likelihood of reaching the maximum strain capacity proposed by Equation (2.6)
- The likelihood of openings of cross-ties (especially smaller ones) at larger drifts even before reaching to fracture strain value which leaves core concrete more vulnerable to abrupt crushing.
- Longitudinal bar buckling phenomena can impose larger transverse strain demands on confining bars thus an equation that does not consider bar buckling in the estimation of compressive strain limits may not yield reasonably.

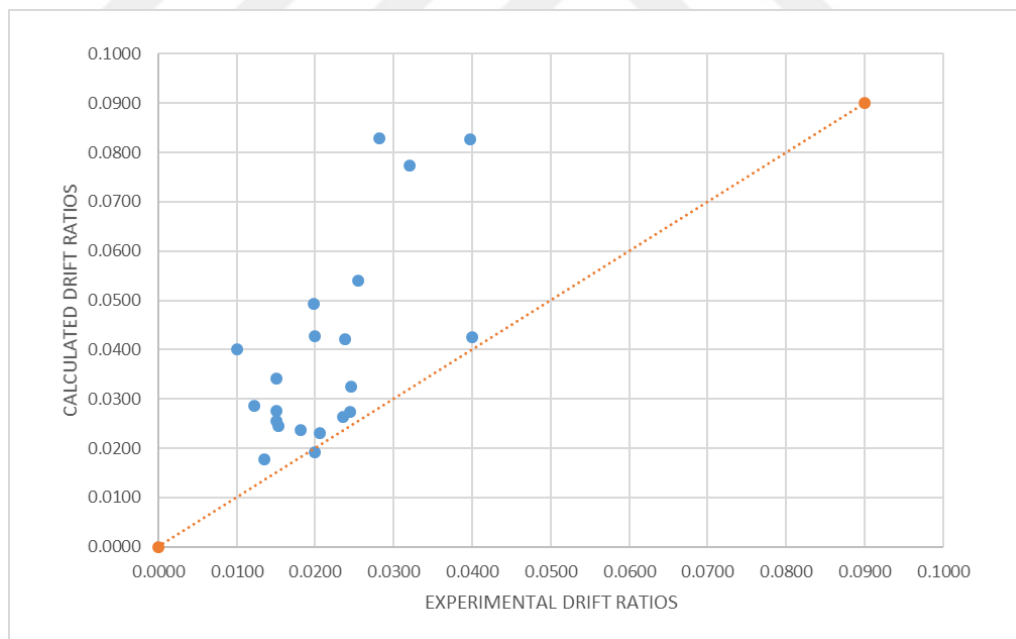


Figure 4.6 : Comparison of experimental and calculated drifts of dataset shearwalls which fail due to CB crushing of concrete.

Figure 4.6 plots the calculated drift values against the experimental values of specimens of which failure mode is CB. Lateral displacements are determined on the basis of the PHA method with the failure criterion as compressive strain limits by Mander et al. (1988). For plastic hinge length estimations, the expression suggested by Eurocode 8 (equation 4.1) is employed. The reason to choose this estimation is the fact that it approximately yields the mean value of 11 different plastic hinge estimations from literature. It can be concluded from the graph that the PHA method with compression side strain limit by Mander et al. (1988) tends to overestimate the actual displacement. There is a need to develop a new compression strain limit expression for shearwalls that accounts also for the impact of buckling of longitudinal bars in confined region and the type of confinement of boundary zones (overlapping hoops or single perimeter hoop +cross tie combination).

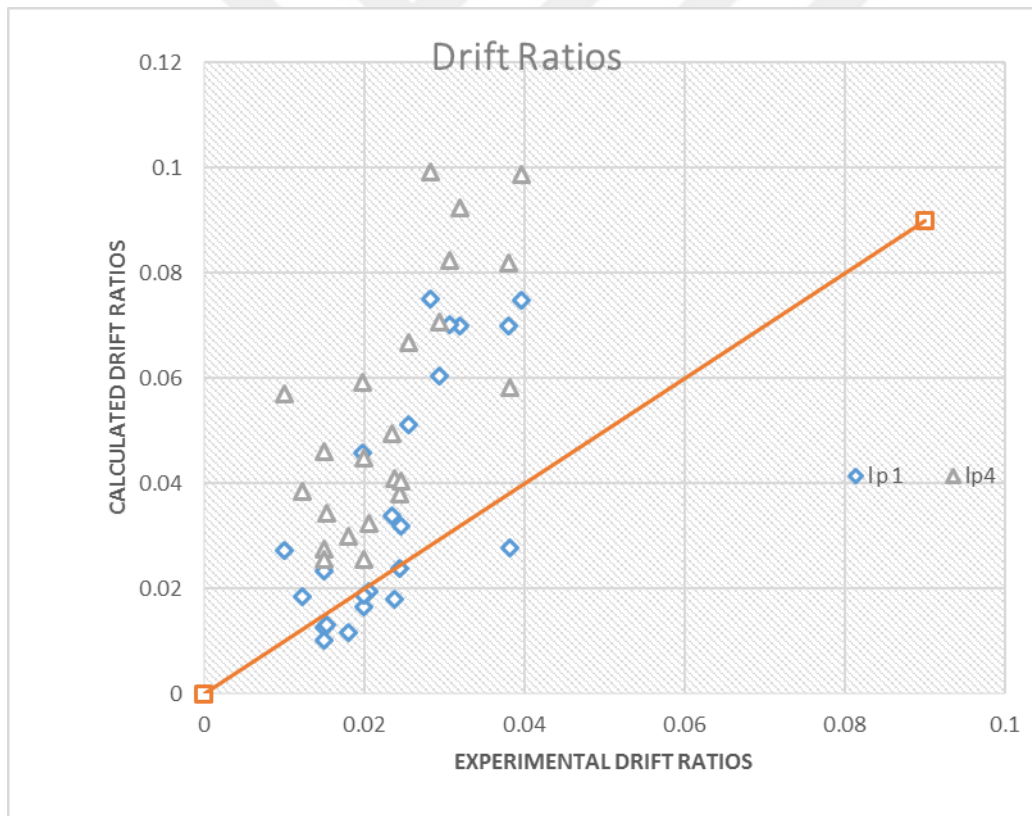


Figure 4.7 : Comparison of experimental and calculated drifts with two different plastic hinge length estimations.

In Figure 4.7 drift values are calculated via PHA method combining compressive strain limits with two different estimation of plastic hinge length estimations. l_{p1} stands for the plastic hinge expression of Paulay and Priestley (1992) which generally

yields the lower-bound values for plastic hinge length while l_{p2} denotes for the classical plastic hinge length approach. ($l_p = 0.5l_w$) which gives the largest plastic hinge length for most the dataset walls. As it can be seen in Figure 4.7, even with the minimum plastic hinge length value, the ultimate strains for compression side tend to overestimate the lateral displacement capacity. The prediction with a plastic hinge length of $0.5l_w$ is more noticable as means of overestimating the capacity.

4.4 The Comparison of the Plastic Hinge Length Estimations

The plastic hinge length is of crucial importance for the assessment procedure by PHA method. The quality of the prediction highly depends on the accuracy of the plastic hinge estimation. Plastic hinge length l_p represent the length over which the plasticity is assumed to extent in lumped plasticity models. The variables which effect the distribution of plasticity has been discussed in Chapter 2. In table 4.1, different proposals from various research groups are presented. In table 4.2, the plastic hinge estimations for dataset shear walls are listed as normalised to the wall length.

Table 4.1 : PHL " l_p " estimations employed in this study

The Plastic Hinge Length
$l_{p1} = 0.08h_w + 0.022f_y d_{bl}$
$l_{p2} = 0.2l_w + 0.044h_w$
$l_{p3} = kh_w + 0.2l_w 0.022f_y d_{bl}$
$l_{p4} = 0.5l_w$
$l_{p5} = 0.12h_w + 0.014f_y d_{bl}$
$l_{p6} = 0.33l_w$
$l_{p7} = 0.2l_w + 0.05h_w(1 - 1.5P/A_g f'_c)$
$l_{p8} = 0.143l_w + 0.072 h_w$
$l_{p9} = 0.2l_w + 0.05h_w(1 - 1.5P/A_g f'_c) 5.1 \Delta_p^{0.3}$
$l_{p10} = 0.27 lw(1 - P/A_g f'_c)(1 - \rho_v (f_{yv}/f_c))(h_w/l_w)^{0.45}$
$l_{p11} = \frac{L_v}{30} + 0.2l_w + 0.11 \frac{f_y d_{bl}}{\sqrt{f'_c}}$

Table 4.2 : PHL estimations normalized to wall length l_w .

Ref.	specimen	l_{p1}	l_{p2}	l_{p3}	l_{p4}	l_{p5}	l_{p6}	l_{p7}	l_{p8}	l_{p9}	l_{p10}	l_{p11}	Average	Average/ l_{p11}
Dazio et.al (2009)	WSH1	0.24	0.30	0.32	0.50	0.31	0.33	0.31	0.31	0.35	0.24	0.32	0.32	1.00
	WSH2	0.18	0.30	0.39	0.50	0.31	0.33	0.30	0.31	0.35	0.24	0.33	0.32	0.99
	WSH3	0.26	0.30	0.37	0.50	0.32	0.33	0.30	0.31	0.35	0.24	0.34	0.33	0.97
	WSH4	0.26	0.30	0.35	0.50	0.32	0.33	0.30	0.31	0.35	0.25	0.34	0.33	0.98
	WSH5	0.23	0.30	0.35	0.50	0.31	0.33	0.29	0.31	0.33	0.22	0.32	0.32	1.00
	WSH6	0.26	0.30	0.35	0.50	0.32	0.33	0.29	0.31	0.33	0.22	0.33	0.32	0.97
Kono et.al 2011	NC80	0.18	0.28	0.36	0.50	0.24	0.33	0.27	0.27	0.30	0.23	0.29	0.30	1.02
	NC40	0.18	0.28	0.36	0.50	0.24	0.33	0.27	0.27	0.30	0.23	0.29	0.30	1.02
	BC40	0.18	0.28	0.36	0.50	0.24	0.33	0.27	0.27	0.30	0.23	0.29	0.30	1.02
	BC80	0.18	0.28	0.36	0.50	0.24	0.33	0.27	0.27	0.30	0.23	0.29	0.30	1.02
Kono et.al 2012	MC	0.16	0.26	0.35	0.50	0.20	0.33	0.26	0.24	0.28	0.23	0.29	0.28	0.97
	SC	0.16	0.26	0.35	0.50	0.20	0.33	0.26	0.24	0.28	0.23	0.29	0.28	0.97
	HN	0.16	0.26	0.35	0.50	0.20	0.33	0.25	0.24	0.27	0.21	0.29	0.28	0.95
	BC	0.18	0.27	0.36	0.50	0.22	0.33	0.27	0.26	0.31	0.25	0.30	0.30	1.00
	NC	0.18	0.27	0.36	0.50	0.22	0.33	0.27	0.26	0.31	0.25	0.30	0.30	1.00
Hines et.al 2002	1A	0.43	0.38	0.63	0.50	0.55	0.33	0.37	0.43	0.45	0.23	0.42	0.43	1.02
	1B	0.43	0.38	0.63	0.50	0.55	0.33	0.37	0.43	0.45	0.23	0.42	0.43	1.03
	2A	0.27	0.29	0.47	0.50	0.31	0.33	0.29	0.29	0.33	0.24	0.35	0.33	0.94
	2B	0.27	0.29	0.47	0.50	0.31	0.33	0.29	0.29	0.32	0.23	0.35	0.33	0.95
	2C	0.29	0.29	0.49	0.50	0.33	0.33	0.29	0.29	0.33	0.23	0.38	0.34	0.90
Oh et.al 2002	WR20	0.21	0.27	0.40	0.50	0.23	0.33	0.27	0.25	0.30	0.24	0.32	0.30	0.93
	WR10	0.21	0.27	0.40	0.50	0.23	0.33	0.27	0.25	0.30	0.24	0.32	0.30	0.94
	WB	0.17	0.27	0.31	0.50	0.21	0.33	0.27	0.25	0.30	0.24	0.29	0.29	0.97

Table 4.2 (continued): PHL estimations normalized to wall length l_w .

Ref.	specimen	l_{p1}	l_{p2}	l_{p3}	l_{p4}	l_{p5}	l_{p6}	l_{p7}	l_{p8}	l_{p9}	l_{p10}	l_{p11}	Average	Average/ l_{p11}
Deng et.al 2008	HPCW01	0.28	0.29	0.48	0.50	0.32	0.33	0.30	0.29	0.35	0.25	0.34	0.34	0.99
	HPCW02	0.28	0.29	0.48	0.50	0.32	0.33	0.30	0.29	0.35	0.26	0.34	0.34	1.01
	HPCW03	0.28	0.29	0.48	0.50	0.32	0.33	0.30	0.29	0.35	0.26	0.34	0.34	1.01
	HPCW04	0.28	0.29	0.48	0.50	0.32	0.33	0.30	0.29	0.35	0.26	0.33	0.34	1.03
1*	RW1	0.31	0.33	0.51	0.50	0.41	0.33	0.33	0.36	0.38	0.23	0.37	0.37	1.01
	RW2	0.31	0.33	0.51	0.50	0.41	0.33	0.33	0.36	0.39	0.24	0.36	0.37	1.04
Segura 2017	WP1	0.36	0.36	0.56	0.50	0.49	0.33	0.36	0.41	0.43	0.23	0.38	0.40	1.07
	WP2	0.36	0.36	0.56	0.50	0.49	0.33	0.36	0.41	0.44	0.24	0.37	0.40	1.08
	WP3	0.36	0.36	0.56	0.50	0.49	0.33	0.36	0.41	0.44	0.24	0.37	0.40	1.08
	WP4	0.57	0.46	0.72	0.50	0.77	0.33	0.46	0.57	0.58	0.24	0.47	0.51	1.10
	WP5	0.36	0.36	0.54	0.50	0.48	0.33	0.36	0.40	0.43	0.24	0.37	0.40	1.06
	WP6	0.36	0.36	0.54	0.50	0.48	0.33	0.36	0.40	0.43	0.24	0.37	0.40	1.06
	WP7	0.36	0.36	0.54	0.50	0.48	0.33	0.36	0.40	0.43	0.24	0.37	0.40	1.06
Birely 2011	PW1	0.28	0.33	0.37	0.50	0.37	0.33	0.32	0.35	0.38	0.23	0.34	0.34	1.02
	PW2	0.21	0.29	0.33	0.50	0.27	0.33	0.28	0.29	0.32	0.22	0.31	0.31	0.99
	PW3	0.19	0.29	0.39	0.50	0.26	0.33	0.28	0.29	0.32	0.23	0.29	0.31	1.04
	PW4	0.20	0.29	0.40	0.50	0.27	0.33	0.28	0.29	0.32	0.22	0.31	0.31	1.01
Oesterle et. al 1976-1979	R1	0.25	0.31	0.45	0.50	0.32	0.33	0.32	0.32	0.37	0.26	0.32	0.34	1.06
	R2	0.26	0.31	0.46	0.50	0.33	0.33	0.32	0.32	0.37	0.26	0.33	0.34	1.05
	B1	0.26	0.31	0.46	0.50	0.33	0.33	0.32	0.32	0.37	0.26	0.33	0.34	1.06
	B3	0.26	0.31	0.46	0.50	0.33	0.33	0.32	0.32	0.37	0.26	0.33	0.34	1.05
	B4	0.26	0.31	0.46	0.50	0.33	0.33	0.32	0.32	0.37	0.26	0.33	0.34	1.04
2*	W1	0.42	0.34	0.52	0.50	0.48	0.33	0.34	0.37	0.40	0.24	0.45	0.40	0.89

1* Thomsen and Wallace (1995), 2* Chen (2005)

Table 4.2 presents the plastic hinge lengths as normalised to wall length l_w . It is noteworthy to state that the average of all estimations for one specimen varies between $0.31l_w$ and $0.51l_w$ and mean value of total table gives $0.34l_w$. At first sight, one can argue that, these are just statistical values and does not represent for the general approaches of the plastic hinge length. However, it basically indicates that the widely accepted approach to use the equivalent plastic hinge length as the half of the wall length may lead unconservative predictions of lateral capacity for most of the specimens discussed herein. The above results also reveal that the estimation of plastic hinge length by Eurocode 8 (CEN 2005) yields approximately the average value of the rest of the estimations.



5. CONCLUSIONS

The study reveals that the closer prediction of lateral capacity strongly depends on three distinct phenomenas; i.) right failure criterion ii.) proper strain limits iii.) a plastic hinge estimation which can be employed in conjunction with the chosen strain limits.

In this study an expression is derived from the section analysis results of dataset walls all of which experienced flexural type failure mode. The aim of this expression is to determine the likely flexural failure mode which is whether crushing of core or rupture of previously buckled bars. Results have shown that unless one adopt right failure criterion, prediction with PHA method may yield irrelevant results far from the actual values. The result of the above mentioned expression “m” value eliminates the vagueness and gives more or less an opinion on whether to chose concrete or steel strain limits as an indicator of final damage state. It is anticipated that c/b ratio is the most profound parameter that identifies the failure mode. Besides, along with the c/l_w , c/b directly effects the drift capacity of shear walls. An expression including these terms would be developed to predict the lateral drift capacity of shear walls.

One important outcome of the study is that confined concrete strain capacity proposed by Mander et al. (1988) would overestimate the drift capacity of a shear wall. These finding may be attributed to the following reasons;

- Buckling of vertical reinforcement bars can impose larger transverse strain demands than anticipated which lead to premature fracture or opening of hoops/cross ties. In other words, the fact that transverse strain demands for one or two cross-ties can disproportionately increase which does not allow redistribution of strain demands to adjacent hoop/tie can cause premature rupture of confining bars leading to abrupt crushing of concrete even before reaching estimated strain capacity.

- Tensile rupture of vertical bars in tension side under cyclic load effects, which does not cause a significant drop in capacity may expedite crushing of confined concrete in compression as longitudinal bars do not contribute to compression resistance anymore. As a matter of fact, tensile rupture mechanism is mostly associated with buckling phenomena so in a way this reason also is indirectly related to buckling. Therefore an equation that does not consider bar buckling in the estimation of compressive strain limits may not yield favorably.
- Ultimate strains obtained from material tests shall not be utilized directly as strain limits in plastic hinge analysis on the ground that these strain limits are only representatives of the mechanical properties and does not take into account the assumptions made by the plane section plane hypothesis and the differences between average axial strains and maximum local strains.
- Besides compressive concrete strain capacity have been derived for pure axial instead of combined axial-flexural loading.

The back calculation of plastic hinge length from experimentally measured curvatures demonstrate that the equation of Eurocode-8 is better suited for assessment purposes, which accounts for all three phenomena (tension-shift, moment gradient strain penetration) that primarily effects the spread of plasticity. The plastic hinge estimation of Eurocode-8 also approximately equals to the average value of the 10 different plastic hinge equation.

When capturing the post-elastic behavior of shear walls by utilization of the plastic hinge method, the plastic hinge length estimation should better consider the reduced spread of plasticity caused by limited flexure-shear cracking and limited hardening. Therein, the preferred strain limits denoting for specific limit states in the scheme of plastic hinge analysis should consider the inequality between maximum and average local strains and the differences between actual strains measured on the wall and strains obtained from plane section analysis.

It is clear that further studies are required to address these issues in terms deriving proper strain limits available to employ in PHA method and establishing better framework of plastic hinge analysis by choosing accurate plastic hinge length estimations in order to increase the quality of performance evaluation methods of shear walls.

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APPENDICES

APPENDIX A: Shear Wall Database

Ref.	Specimen	Basic Geometry of Specimen								Rebar Quantities									
										Boundary Zone									
		aspect ratio	b _w mm	l _w mm	L _v mm	M _B /V _U	H _w mm	Boundary Zone			Longitudinal					Confining reinf.			
								b _{bw}	l _{boundary}	cc	no bars	Xline	Yline	dia	ρ _b	type* ¹	ρ _{vol} (x+y)	Hoops	
Dazio et.al. 2009	WSH1	2.28	150	2000	4560	4560	150	150	25	6	3	2	Ø 10.00	0.0157	1	0.01331	Ø 6.00	75.00	
	WSH2	2.28	150	2000	4560	4560	150	150	25	6	3	2	Ø 10.00	0.0157	1	0.01331	Ø 6.00	75.00	
	WSH3	2.28	150	2000	4560	4560	150	200	25	6	3	2	Ø 12.00	0.0181	1	0.01158	Ø 6.00	75.00	
	WSH4	2.28	150	2000	4560	4560	150	200	25	6	3	2	Ø 12.00	0.0181	no confining bars				
	WSH5	2.28	150	2000	4560	4560	150	200	25	6	3	2	Ø 8.00	0.0080	3	0.01263	Ø 4.20	50.00	
	WSH6	2.28	150	2000	4560	4520	150	325	25	8	4	2	Ø 12.00	0.0161	3	0.01749	Ø 5.47	50.00	
Kono et.al 2011	NC80	1.71	128	1750	3000	2800	128	490	25	12	6	2	Ø 10.00	0.0136	9	0.01687	Ø 6.00	80.00	
	NC40	1.71	128	1750	3000	2800	128	490	25	12	6	2	Ø 10.00	0.0136	9	0.03373	Ø 6.00	40.00	
	BC40	1.71	80	1750	3000	2800	250	200	25	8	3	3	Ø 10.00	0.0101	2	0.01963	Ø 6.00	40.00	
	BC80	1.71	80	1750	3000	2800	250	200	25	8	3	3	Ø 10.00	0.0101	0	0.00654	Ø 6.00	80.00	
Kono et.al 2012	MC	1.37	120	1750	2400	1700	120	200	25	10	5	2	Ø 10.00	0.0262	10	0.01188	Ø 4.00	80.00	
	SC	1.37	120	1750	2400	1700	120	100	25	6	3	2	Ø 10.00	0.0262	0	0.01299	Ø 4.00	40.00	
	HN	1.37	120	1750	2400	1700	120	400	25	10	5	2	Ø 10.00	0.0145	11	0.02255	Ø 4.00	40.00	
	BC	1.60	91	1500	2400	1700	200	152	24	8	3	3	Ø 10.00	0.0157	12	0.01136	Ø 4.00	40.00	
	NC	1.60	120	1500	2400	1700	120	151	19	8	4	2	Ø 10.00	0.0277	3	0.01743	Ø 4.00	40.00	

Ref.	Specimen	Basic Geometry of Specimen								Rebar Quantities								
										Boundary Zone								
		aspect ratio	Boundary Zone							Longitudinal				Confining reinforcement.				
			b _w mm	l _w mm	L _v M _B /V _U	H _w mm	b _{bw}	l _{boundary}	CC	no bars	Xline	Yline	dia	ρ _b	type* ¹	ρ _{vol} (x+y)	Hoops	
																	dia	space. (mm)
Hines 2004	1A	4.04	152	1219	4928	4877	305	305		9			Ø 13.00	0.0129	Spiral	0.01339	Ø 10.00	76.20
	1B	4.04	152	1219	4928	4877	305	305		9			Ø 13.00	0.0129	Spiral	0.01339	Ø 10.00	76.20
	2A	2.00	152	1219	2438	2438	305	305		9			Ø 13.00	0.0129	Spiral	0.01339	Ø 10.00	76.20
	2B	2.00	152	1219	2438	2438	305	305		9			Ø 13.00	0.0129	Spiral	0.01339	Ø 10.00	76.20
	2C	2.09	102	1168	2438	2438	305	305		9			Ø 13.00	0.0129	Spiral	0.01339	Ø 10.00	76.20
oh et al 2002	WR20	1.50	200	1500	2250	2000	200	100	50	4	2	2	Ø 13.00	0.0133	0	0.01277	Ø 10.00	200.00
	WR10	1.50	200	1500	2250	2000	200	100	50	4	2	2	Ø 13.00	0.0133	0	0.02554	Ø 10.00	100.00
	WB	1.50	125	1500	2250	2000	240	180	40	8	3	3	Ø 10.00	0.0101	0	0.01105	Ø 10.00	150.00
deng 2008	HPCW01	2.10	100	1000	2100	2000	100	160	10	6	3	2	Ø 12.00	0.0377	2	0.00912	Ø 4.00	60.00
	HPCW02	2.10	100	1000	2100	2000	100	240	10	8	4	2	Ø 12.00	0.0348	13	0.01439	Ø 5.00	60.00
	HPCW03	2.10	100	1000	2100	2000	100	240	10	8	4	2	Ø 12.00	0.0348	13	0.02158	Ø 5.00	40.00
	HPCW04	2.10	100	1000	2100	2000	100	320	10	11	5	3	Ø 12.00	0.0366	14	0.03116	Ø 6.00	40.00
Thomsen 1995	RW1	3.00	102	1219	3660	3660	102	152	19	8	4	2	Ø 9.53	0.0295	2	0.01161	Ø 4.76	76.20
	RW2	3.00	102	1219	3660	3660	102	152	19	8	4	2	Ø 9.53	0.0295	0	0.01321	Ø 4.76	50.80
Segura 2007	WP1	3.74	152	2286	8560	2134	152	305	25	14	6	2	Ø 12.70	0.0327	2	0.01803	Ø 6.35	50.80
	WP2	3.74	152	2286	8560	2134	152	287	35	14	6	2	Ø 12.70	0.0327	2	0.03158	Ø 7.94	50.80
	WP3	3.74	152	2286	8560	2134	152	305	25	14	6	2	Ø 12.70	0.0327	2	0.01862	Ø 7.94	76.20
	WP4	5.86	152	2286	13400	2134	152	305	25	14	6	2	Ø 19.05	0.0736	1	0.00688	Ø 6.35	114.30
	WP5	3.57	152	2286	8170	2134	152	305	25	10	5	2	Ø 15.88	0.0365	4	0.04111	Ø 7.94	50.80
	WP6	3.57	191	2286	8170	2134	152	305	25	10	4	3	Ø 15.88	0.0365	3	0.02273	Ø 6.35	50.80
	WP7	3.57	229	2286	8170	2134	152	305	25	12	4	3	Ø 15.88	0.0438	3	0.02273	Ø 6.35	50.80

Ref.	Specimen	Basic Geometry of Specimen								Rebar Quantities								
										Boundary Zone								
		aspect ratio	Boundary Zone							Longitudinal				ρ_b	Confining reinforcement.			
			b_w mm	l_w mm	L_v	M_B/V_U	H_w mm	b_{bw}	$l_{boundary}$	no bars	Xline	Yline	dia		type* ¹	$\rho_{vol}(x+y)$	Hoops	
																	dia	space. (mm)
lowes 2011	PW1	2.84	152	3048	8662	3660	152	457	38	21	7	3	Ø 12.70	0.0327	17	0.02225	Ø 6.35	50.80
	PW2	2.00	152	3048	6100	3660	152	457	25	21	7	3	Ø 12.70	0.0344	17	0.01950	Ø 6.35	50.80
	PW3	2.00	152	3048	6100	3660	152	457	25	14	7	3	Ø 12.70	0.0229	13	0.02079	Ø 6.35	44.45
	PW4	2.00	152	3048	6100	3660	152	457	25	21	7	3	Ø 12.70	0.0344	17	0.01950	Ø 6.35	50.80
oesterle 1976-1979	R1	2.40	102	1905	4572	4572	102	140	25	4	2	2	Ø 9.53	0.0147	0	0.00820	Ø 4.95	102.00
	R2	2.40	102	1905	4572	4572	102	140	25	6	3	2	Ø 12.70	0.0391	1	0.03954	Ø 6.00	34.00
	B1	2.40	102	1905	4572	4572	305	254	25	8	3	3	Ø 12.70	0.0109	16	0.00211	Ø 4.95	202.00
	B3	2.40	102	1905	4572	4572	305	254	25	8	3	3	Ø 12.70	0.0109	16	0.01829	Ø 6.00	34.00
	B4	2.40	102	1905	4572	4572	305	254	25	8	3	3	Ø 12.70	0.0109	16	0.01829	Ø 6.00	34.00
Liu	W1	3.13	200	1200	3750	3750	200	140	40	4	2	2	Ø 19.50	0.0271	0	0.02504	Ø 11.30	100.00
	W2	3.13	200	1200	3750	3750	200	140	40	4	2	2	Ø 19.50	0.0271	0	0.02504	Ø 11.30	100.00
chen	W3	3.13	200	1200	3750	3750	200	140	40	4	2	2	Ø 19.50	0.0271	0	0.04174	Ø 11.30	60.00
	W4	3.13	200	1200	3750	3750	200	140	40	4	2	2	Ø 19.50	0.0271	0	0.04174	Ø 11.30	60.00

Ref.	Specimen	Material Properties										
		Concrete				Boundary Zone Reinforcement						
						Longitudinal				Transverse		
		E_C Gpa	f'_c Mpa	f'_{cc} Mpa	ϵ_{cu}	f_y Mpa	f_u Mpa	ϵ_{su}	f_u/f_y	f_y Mpa	f_u Mpa	ϵ_{su}
Dazio et al. 2009	WSH1	44.40	45.00	54.92	0.00863	547.30	619.90	0.0464	1.13	583.00	534.50	0.02340
	WSH2	37.10	40.50	48.80	0.01467	583.10	747.40	0.0771	1.28	484.90	534.50	0.05760
	WSH3	35.20	39.20	46.86	0.01491	601.00	725.50	0.0769	1.21	489.00	552.20	0.06450
	WSH4	38.50	40.90	40.90		576.00	674.90	0.0729	1.17			
	WSH5	36.10	38.30	48.56	0.01027	583.70	714.40	0.0785	1.22	562.20	615.00	0.03060
	WSH6	36.90	45.60	61.69	0.01083	576.00	674.90	0.0729	1.17	562.20	615.00	0.03060
Kono et.al 2011	NC80	30.90	52.50	60.88	0.01000	377.00	496.00	0.1500	1.32	387.00	533.00	0.04000
	NC40	30.90	52.50	74.05	0.01387	377.00	496.00	0.1500	1.32	387.00	533.00	0.04000
	BC40	30.10	59.50	67.38	0.01032	377.00	496.00	0.1500	1.32	387.00	533.00	0.04000
	BC80	30.10	59.50	61.70	0.00630	377.00	496.00	0.1500	1.32	387.00	533.00	0.04000
Kono et.al 2012	MC	33.00	29.60	35.21	0.01147	385.00	524.00	0.1500	1.36	395.00	524.00	0.04000
	SC	29.50	27.50	35.64	0.01206	385.00	524.00	0.1500	1.36	395.00	524.00	0.04000
	HN	31.00	27.80	40.68	0.01626	385.00	524.00	0.1500	1.36	395.00	524.00	0.04000
	BC	28.20	35.20	39.88	0.01012	348.00	467.00	0.1500	1.34	384.00	523.00	0.04000
	NC	28.70	35.50	47.31	0.01192	348.00	467.00	0.1500	1.34	384.00	523.00	0.04000

Ref.	Specimen	Material Properties										
		Concrete				Boundary Zone Reinforcement						
						Longitudinal				Transverse		
		E_C Gpa	$f_{c'}$ Mpa	$f_{cc'}$ Mpa	ϵ_{cu}	f_y Mpa	f_u Mpa	ϵ_{su}	f_u/f_y	f_y Mpa	f_u Mpa	ϵ_{su}
Hines 2004	1A		38.10	55.64	0.01957	462.00	690.00	0.1180	1.49	462.00	738.00	0.10000
	1B		42.80	60.60	0.01843	462.00	690.00	0.1180	1.49	462.00	738.00	0.10100
	2A		36.60	53.99	0.02164	455.00	662.00	0.1000	1.45	503.00	745.00	0.10100
	2B		41.50	59.23	0.02008	455.00	662.00	0.1000	1.45	503.00	745.00	0.10100
	2C		31.10	48.06	0.02354	496.00	724.00	0.1000	1.46	496.00	703.00	0.10100
oh et al 2002	WR20		34.20	34.73	0.01280	449.00	617.00	0.1300	1.37	342.00	445.00	0.05000
	WR10		36.20	43.80	0.01796	449.00	617.00	0.1300	1.37	342.00	445.00	0.05000
	WB		33.70	36.07	0.01134	342.00	415.00	0.1300	1.21	342.00	445.00	0.05000
deng 2008	HPCW01		61.43	72.25	0.01139	433.60	636.60	0.0800	1.47	836.00	836.00	0.05000
	HPCW02		73.56	89.49	0.01202	433.60	636.60	0.0800	1.47	713.00	713.00	0.05000
	HPCW03		75.32	102.39	0.01452	433.60	636.60	0.0800	1.47	713.00	713.00	0.05000
	HPCW04		86.02	118.22	0.01509	433.60	636.60	0.0800	1.47	601.00	601.00	0.05000
Thomsen 1995	RW1		31.58	37.14	0.01539	434.37	641.21	0.1000	1.48	434.00	483.00	0.06000
	RW2		43.64	52.59	0.01316	434.37	641.21	0.1000	1.48	434.00	483.00	0.06000

Ref.	Specimen	Material Properties										
		Concrete				Boundary Zone Reinforcement						
		E_C Gpa	$f_{c'}$ Mpa	$f_{cc'}$ Mpa	ϵ_{cu}	Longitudinal				Transverse		
						f_y Mpa	f_u Mpa	ϵ_{su}	f_u/f_y	f_y Mpa	f_u Mpa	ϵ_{su}
Segura 2007	WP1		35.78	47.66	0.01453	506.07	743.25	0.1110	1.47	337.00	403.00	0.05900
	WP2		41.71	64.50	0.01987	506.07	743.25	0.1110	1.47	406.00	481.00	0.05700
	WP3		42.33	56.41	0.01596	506.07	743.25	0.1110	1.47	454.00	706.00	0.05700
	WP4		45.92	49.25	0.00789	530.20	722.56	0.1380	1.36	337.00	403.00	0.05900
	WP5		47.92	79.49	0.02657	488.83	669.48	0.1430	1.37	439.00	531.00	0.07100
	WP6		46.26	64.16	0.02622	488.83	669.48	0.1430	1.37	487.00	598.00	0.09200
	WP7		48.54	66.54	0.02543	488.83	669.48	0.1430	1.37	487.00	598.00	0.09200
lowes 2011	PW1		36.07	54.74	0.01885	579.15	694.99	0.0860	1.20	521.93	583.29	0.05000
	PW2		40.29	58.26	0.01623	579.15	694.99	0.0860	1.20	521.93	583.29	0.05000
	PW3		34.34	53.46	0.01821	353.70	537.10	0.1400	1.52	521.93	583.29	0.05000
	PW4		29.45	46.56	0.01930	462.63	641.21	0.0940	1.39	521.93	583.29	0.05000
oesterle 1976 -1979	R1	27.79	44.70	45.59	0.01076	511.60	765.30	0.0980	1.50	455.00	496.44	0.05900
	R2	26.82	46.40	78.36	0.04558	450.24	708.11	0.1230	1.57	535.05	695.00	0.11000
	B1	28.13	53.00	54.07	0.00684	450.22	708.09	0.1170	1.57	473.67	517.79	0.11000
	B3	27.30	47.30	65.44	0.02392	437.81	696.36	0.1090	1.59	475.73	658.44	0.10700
	B4		45.02	64.00	0.02944	450.22	696.36	0.1180	1.55	504.69	681.20	0.12600
Liu 2004	W1		33.10	45.59	0.00928	464.40	580.90	0.1700	1.25	457.70	630.30	0.01500
	W2		70.64	84.02	0.00686	464.40	580.90	0.1700	1.25	457.70	630.30	0.01500
Chen 2005	W3		28.40	52.50	0.01164	464.40	580.90	0.1700	1.25	457.70	630.30	0.01500
	W4		69.20	97.52	0.00811	464.40	580.90	0.1700	1.25	457.70	630.30	0.01500

APPENDIX B : Section Analysis Results

Ref.	Specimen	failure type	drift	δu	s/db	n	Yield point $\epsilon_y = f_y/E_s$				Nominal point $\epsilon_y = 0.015$ or $\epsilon_c = 0.004$				
							ϵ_{sy}	ϵ_c	c	M_y	ϵ_s	ϵ_c	c	M_N	ϵ_{midde}
Dazio et al. 2009	WSH1	R0	0.01	47.5	7.50	0.051	0.002234	0.00068	455.04	1052	0.015	0.002	229.412	1417	0.0065
	WSH2	BR	0.01	63	7.50	0.057	0.002194	0.00072	481.81	1043	0.0153	0.00217	242.215	1411	0.006565
	WSH3	BR	0.02	92.4	6.25	0.058	0.002123	0.00077	519.01	1210	0.01457	0.00262	297.208	1853	0.005975
	WSH4	CB	0.01	61.6		0.057	0.002169	0.00073	491.03	1257	0.01492	0.00245	275.043	1857	0.006235
	WSH5	BR	0.01	62	6.25	0.128	0.003063	0.0012791	574.43	1557	0.01544	0.00337	349.362	1825	0.006035
	WSH6	CB	0.02	93.7	4.17	0.128	0.00194	0.00094	636.46	1702	0.0155	0.00323	336.279	2468	0.006135
Kono et.al 2011	NC80	CB	0.02	45	8.00	0.112	0.00153	0.00102	680.00	1227	0.0153	0.0037	331.053	1665	0.0058
	NC40	CB	0.02	71.4	4.00	0.112	0.001589	0.000966	642.74	1267	0.0156	0.0034	304.211	1695	0.0061
	BC40	BR	0.04	120	4.00	0.112	0.0018	0.000744	497.17	1454	0.0148	0.00189	192.51	1703	0.006455
	BC80	BR	0.02	60	8.00	0.112	0.0017805	0.0007741	515.16	1436.43	0.01469	0.00197	201.069	1690.96	0.006358
Kono et.al 2012	MC	CB	0.02	36.72	8.00	0.104	0.001659	0.0008956	595.98	792.081	0.01476	0.00313	297.27	1096.46	0.005814
	SC	CB	0.02	43.44	4.00	0.104	0.0016822	0.0008799	583.82	669.995	0.01503	0.00291	275.368	873.038	0.006062
	HN	CB	0.02	36	4.00	0.206	0.0018512	0.0019816	878.92	1025.7	0.00754	0.00395	584.678	1255.98	0.001795
	BC	BR	0.04	91.2	4.00	0.047	0.0016016	0.0005651	378.69	467.244	0.01561	0.00172	144.111	614.48	0.006945
	NC	CB	0.02	48	4.00	0.047	0.0023907	0.0008632	387.83	498.009	0.01581	0.00251	200.359	666.831	0.00665

Ref.	Specimen	failure type	drift	δu	s/db	n	Yield point $\epsilon_y = f_y/E_s$				Nominal point $\epsilon_y = 0.015$ or $\epsilon_c = 0.004$				
							ϵ_{sy}	ϵ_c	c	M_y	ϵ_s	ϵ_c	c	M_N	ϵ_{midde}
Hines 2004	1A	BR	0.05	226	5.86	0.09732	0.0023548	0.0011236	393.82	913	0.01516	0.0031	206.977	1244	0.006031
	1B	BR	0.05	226	5.86	0.09068	0.0023624	0.001116	391.16	914	0.01523	0.00303	202.39	1248.78	0.006099
	2A	BR	0.03	71.1	5.86	0.08072	0.0023815	0.0010969	384.46	924.091	0.01527	0.00299	199.894	1240.38	0.006137
	2B	BR	0.03	71.1	5.86	0.0944	0.0024	0.0010261	365.14	951.104	0.01548	0.00278	185.695	1252.27	0.006349
	2C	BR	0.03	71.1	5.86	0.10892	0.0022393	0.0011629	399.36	795.743	0.01532	0.00339	211.91	1190.37	0.005962
Oh et al 2006	WR20	CB	0.04	80.9	15.38	0.07973	0.002463	0.0006734	300.59	980.12	0.01532	0.00141	117.693	1158.56	0.006958
	WR10	CB	0.04	85.9	7.69	0.07532	0.0024796	0.0006568	293.17	982.835	0.01536	0.00133	111.582	1108.72	0.007014
	WB	CB	0.04	84.7	15.00	0.07532	0.0015168	0.0006297	416.55	691.346	0.01527	0.00191	157.661	897.796	0.006679
Deng 2008	HPCW01	CB	0.02	42	5.00	0.02605	0.0022157	0.0013639	373.41	533.543	0.01383	0.00407	222.71	632.297	0.004882
	HPCW02	BR	0.02	52.08	5.00	0.02175	0.0022725	0.0013065	357.75	555.539	0.01535	0.00397	201.496	654.023	0.00569
	HPCW03	CB	0.02	51.4	3.33	0.02124	0.0022811	0.0012979	355.39	596.02	0.01484	0.00377	198.307	710.194	0.005539
	HPCW04	BR	0.03	56.22	3.33	0.0186	0.0023518	0.0012268	335.96	628.254	0.01531	0.0033	173.967	726.6	0.006001
Thomsen	RW1	BR	0.02	82.72	8.00	0.10237	0.0023803	0.0011245	378.94	448.51	0.01531	0.00309	198.247	530.109	0.006112
	RW2	CB	0.02	86.01	5.33	0.07407	0.0025747	0.00093	313.42	465.074	0.01518	0.00234	157.999	533.317	0.006418

Ref.	Specimen	failure type	drift	δ_u	s/db	n	Yield point $\epsilon_y = f_y/E_s$				Nominal point $\epsilon_y = 0.015$ or $\epsilon_c = 0.004$				
							ϵ_{sy}	ϵ_c	c	M_y	ϵ_s	ϵ_c	c	M_N	ϵ_{midde}
segura 2007	WP1	CB	0.02	209.8	4.00	0.09634	0.0035749	0.0014238	636.66	3017.27	0.01499	0.00335	407.914	3437.11	0.00582
	WP2	CB	0.03	218.2	4.00	0.08264	0.0035879	0.0013699	612.36	3020.58	0.01501	0.00317	386.626	3452.85	0.005919
	WP3	CB	0.02	169.7	6.00	0.08143	0.0035405	0.0014581	652.02	2993.16	0.01498	0.00335	408.49	3446.33	0.005816
	WP4	O	0.04	519.9	6.00	0.07507	0.0027229	0.0006108	409.54	3336.69	0.01546	0.00121	162.481	4043.65	0.007122
	WP5	CB	0.03	230.4	3.20	0.07194	0.0036123	0.0013808	618.14	3466.42	0.01507	0.00326	397.354	3886.64	0.005905
	WP6	CB	0.03	261.1	3.20	0.0745	0.0036444	0.0013536	605.35	3698.05	0.01522	0.00311	379.515	4215.46	0.006053
	WP7	CB	0.04	324.4	3.20	0.07104	0.0036939	0.0013	581.86	4398.74	0.0154	0.00293	356.888	4974.65	0.006238
lowes 2011	PW1	R0	0.02	132.5	5.33	0.09532	0.0029959	0.0014422	965.71	5557	0.01234	0.00319	610.572	6506.29	0.004575
	PW2	CB	0.02	91.5	4.00	0.1298	0.0029137	0.0015625	1046.22	6339.82	0.01596	0.00419	623.1	7593.43	0.005884
	PW3	CB	0.01	74.42	3.50	0.10038	0.0029476	0.00153	1024.16	4380.77	0.01161	0.00406	777.164	5458.54	0.003772
	PW4	CB	0.01	61.61	4.00	0.11702	0.0029307	0.0015455	1034.87	5008.81	0.01601	0.00413	614.324	5787.07	0.005943
oesterle 1976 - 1979	R1	BR	0.02	103.3	10.70	0	0.0023785	0.0004194	277.96	309.30	0.01542	0.00137	150.937	422.994	0.007027
	R2	BR	0.03	133.5	2.68	0	0.0022851	0.0005129	339.89	648.112	0.01531	0.00147	162.714	798.584	0.006921
	B1	BR	0.03	132.1	15.91	0	0.0024218	0.0003547	236.85	821.633	0.01571	0.00095	106.126	1007.94	0.007376
	B3	BR	0.04	179.7	2.68	0	0.0024577	0.0003395	225.04	812.31	0.01588	0.00091	100.222	977.209	0.007484
	B4	R0	0.06	271.6	2.68	0	0.0024577	0.0003395	225.04	812.308	0.01588	0.00091	100.222	978.209	0.007484
Liu 2005	W1	CB	0.03	110.2	5.13	0.07553	0.0023543	0.0010218	338.97	852.39	0.01508	0.00292	181.801	1027.88	0.00608
	W2	BR	0.03	108	5.13	0.03539	0.0024256	0.0009505	315.33	877.46	0.01555	0.00246	153.047	1048.97	0.006542
Chen 2005	W3	CB	0.04	142.3	3.08	0.08803	0.0023126	0.0009505	326.25	841.60	0.01503	0.00297	185.009	1024.5	0.006029
	W4	CB	0.03	114.9	3.08	0.03613	0.0022439	0.0011322	375.60	842.00	0.01548	0.00252	156.946	1048.7	0.00648

APPENDIX C : Plastic Hinge Analysis Results with Compressive Strain Limits

Ref.	Specimen	failure type	drift	l _{p1}			l _{p2}			l _{p3}			l _{p4}			l _{p5}		
				Δ_f	Δ_u	cal/exp	Δ_f	Δ_u	cal/exp	Δ_f	Δ_u	cal/exp	Δ_f	Δ_u	cal/exp	Δ_f	Δ_u	cal/exp
Dazio et al. 2009	WSH1	R0	0.01042	103.885	119.615	2.51821	123.792	142.537	3.00077	130.683	150.47	3.16779	188.252	216.757	4.5633	127.721	147.06	3.09601
	WSH2	BR	0.01382	100.317	115.246	1.82931	155.453	178.587	2.83472	194.921	223.928	3.55441	238.458	273.944	4.34832	161.603	185.652	2.94685
	WSH3	BR	0.02026	108.075	122.951	1.33064	120.736	137.355	1.48653	144.168	164.012	1.77503	182.348	207.448	2.24511	128.412	146.087	1.58103
	WSH4	CB	0.01351	58.6644	67.0035	1.08772	65.2141	74.4842	1.20916	73.4564	83.8981	1.36198	94.5512	107.992	1.75311	68.5477	78.2916	1.27097
	WSH5	BR	0.0136	66.762	75.2445	1.21362	79.6572	89.7781	1.44803	89.6493	101.04	1.62967	115.787	130.499	2.10482	80.7925	91.0577	1.46867
	WSH6	CB	0.02055	78.0025	88.1204	0.94045	87.5639	98.9221	1.05573	99.5964	112.515	1.2008	130.391	147.305	1.57209	92.4304	104.42	1.11441
Kono et al 2011	NC80	CB	0.015	32.2315	37.3946	0.83099	44.3613	51.4676	1.14372	54.446	63.1678	1.40373	71.1464	82.5435	1.8343	39.1739	45.4492	1.00998
	NC40	CB	0.0238	45.9278	53.6656	0.75162	64.26	75.0864	1.05163	79.5015	92.8957	1.30106	104.741	122.388	1.71412	56.4201	65.9256	0.92333
	BC40	BR	0.04	72.3964	87.0898	0.72575	102.962	123.859	1.03216	128.375	154.43	1.28691	170.458	205.054	1.70878	89.8907	108.135	0.90112
	BC80	BR	0.02	50.8457	61.0309	1.01718	71.6792	86.0376	1.43396	89.0002	106.828	1.78047	117.684	141.258	2.35429	62.7696	75.3433	1.25572
Kono et al 2012	MC	CB	0.0153	25.9456	31.4792	0.85728	39.866	48.3683	1.31722	50.7584	61.5839	1.67712	67.6998	82.1385	2.23689	31.1607	37.8065	1.02959
	SC	CB	0.0181	22.6667	27.6925	0.63749	34.7458	42.4498	0.97721	44.1975	53.9972	1.24303	58.898	71.9572	1.65647	27.192	33.2212	0.76476
	HN	CB	0.015	21.9706	24.2198	0.67277	33.1986	36.5973	1.01659	41.9843	46.2825	1.28562	55.649	61.3461	1.70406	26.177	28.8569	0.80158
	BC	BR	0.038	46.5695	57.0255	0.62528	67.3274	82.4442	0.90399	86.4415	105.85	1.16063	113.408	138.871	1.5227	57.0663	69.8791	0.76622
	NC	CB	0.02	37.4618	45.1322	0.94025	53.533	64.4939	1.34362	68.3313	82.3222	1.71505	89.2089	107.475	2.23905	45.5886	54.923	1.14423
Hines 2004	1A	BR	0.04586	195.125	209.888	0.92871	175.629	188.917	0.83592	264.954	285.001	1.26107	219.406	236.007	1.04428	238.288	256.317	1.13414
	1B	BR	0.04586	212.843	229.13	1.01385	191.256	205.891	0.91102	290.168	312.372	1.38218	239.731	258.076	1.14193	260.639	280.584	1.24152
	2A	BR	0.02916	65.0214	75.1402	1.05682	69.2499	80.0268	1.12555	102.705	118.689	1.66932	108.53	125.42	1.76399	73.1567	84.5416	1.18905
	2B	BR	0.02916	64.1658	74.4975	1.04779	68.3502	79.3557	1.11611	101.457	117.793	1.65672	107.221	124.485	1.75084	72.2163	83.8443	1.17924
	2C	BR	0.02916	66.464	75.8628	1.06699	67.108	76.5978	1.07733	101.536	115.895	1.63003	103.454	118.083	1.66081	73.6846	84.1044	1.1829
oh et al 2010	WR20	CB	0.03596	37.7932	46.8509	0.57912	46.8986	58.1386	0.71865	65.268	80.9105	1.00013	78.0721	96.7833	1.19633	42.2002	52.3142	0.64665
	WR10	CB	0.03818	50.1915	62.3469	0.72581	62.6347	77.8036	0.90575	87.7378	108.986	1.26876	105.235	130.722	1.52179	56.214	69.828	0.8129
	WB	CB	0.03764	38.2491	46.9443	0.55424	56.4317	69.2604	0.81771	65.0615	79.852	0.94276	95.1144	116.737	1.37824	46.3388	56.8731	0.67147

Ref.	Specimen	failure type	drift	l _{p6}			l _{p7}			l _{p8}			l _{p9}			l _{p10}		
				Δ _f	Δ _u	cal/exp	Δ _f	Δ _u	cal/exp	Δ _f	Δ _u	cal/exp	Δ _f	Δ _u	cal/exp	Δ _f	Δ _u	cal/exp
Dazio et al. 2009	WSH1	R0	0.01042	133.807	154.067	3.24352	125.476	144.475	3.04159	126.114	145.209	3.05704	141.236	162.622	3.42361	104.445	120.26	2.53178
	WSH2	BR	0.01382	168.349	193.402	3.06987	157.174	180.563	2.86609	158.443	182.021	2.88923	177.283	203.665	3.23278	130.253	149.637	2.37519
	WSH3	BR	0.02026	130.308	148.245	1.60438	121.957	138.745	1.50156	122.955	139.879	1.51385	136.861	155.7	1.68506	101.789	115.801	1.25325
	WSH4	CB	0.01351	69.7719	79.6898	1.29367	65.822	75.1785	1.22043	66.2706	75.6908	1.22875	72.9294	83.2962	1.35221	58.06	66.3132	1.07651
	WSH5	BR	0.0136	85.2703	96.1044	1.55007	78.0899	88.0116	1.41954	80.9583	91.2445	1.47169	85.8825	96.7944	1.5612	64.6203	72.8307	1.17469
	WSH6	CB	0.02055	94.2175	106.439	1.13595	85.7061	96.8233	1.03333	89.1062	100.664	1.07433	94.9432	107.259	1.1447	70.2099	79.3171	0.8465
Kono et al. 201	NC80	CB	0.015	51.2872	59.5029	1.32229	43.8283	50.8492	1.12998	43.1933	50.1125	1.11361	47.9162	55.5919	1.23538	38.9417	45.1798	1.004
	NC40	CB	0.0238	74.7274	87.3173	1.22293	63.4545	74.1451	1.03845	62.4949	73.0238	1.02274	69.6326	81.3641	1.13955	56.0693	65.5156	0.91759
	BC40	BR	0.04	120.415	144.854	1.20712	101.619	122.244	1.0187	100.019	120.319	1.00266	111.92	134.635	1.12196	89.5278	107.698	0.89748
	BC80	BR	0.02	83.5747	100.316	1.67193	70.7637	84.9387	1.41565	69.6732	83.6298	1.39383	77.7848	93.3663	1.55611	62.5223	75.0464	1.25077
Kono et al. 2012	MC	CB	0.0153	48.6497	59.0254	1.60745	39.5443	47.978	1.30659	37.4244	45.406	1.23655	42.8785	52.0234	1.41676	36.2115	43.9345	1.19647
	SC	CB	0.0181	42.3677	51.7617	1.19157	34.4667	42.1088	0.96935	32.6272	39.8614	0.91762	37.3599	45.6435	1.05073	31.4925	38.4751	0.88571
	HN	CB	0.015	40.2834	44.4075	1.23354	31.8298	35.0884	0.97468	31.2292	34.4263	0.95629	34.0573	37.544	1.04289	27.3091	30.1049	0.83625
	BC	BR	0.038	80.126	98.1163	1.07584	68.196	83.5078	0.91566	64.6352	79.1475	0.86785	75.3806	92.3055	1.01212	62.5084	76.5431	0.83929
	NC	CB	0.02	63.4418	76.4315	1.59232	54.2054	65.304	1.3605	51.4486	61.9828	1.29131	59.7679	72.0054	1.50011	49.8146	60.0142	1.2503
Hines 2004	1A	BR	0.04586	158.077	170.037	0.75238	173.714	186.857	0.8268	195.946	210.771	0.93262	201.562	216.812	0.95935	120.609	129.735	0.57405
	1B	BR	0.04586	171.82	184.967	0.81844	189.948	204.484	0.90479	213.753	230.109	1.01818	221.123	238.044	1.05329	131.684	141.76	0.62726
	2A	BR	0.02916	77.446	89.4985	1.25877	69.2285	80.0021	1.12521	69.0505	79.7964	1.12231	76.8941	88.8606	1.2498	58.94	68.1124	0.95798
	2B	BR	0.02916	76.4609	88.7723	1.24856	67.928	78.8655	1.10922	68.1529	79.1267	1.11289	75.3476	87.4798	1.23038	57.7119	67.0045	0.9424
	2C	BR	0.02916	74.1103	84.5903	1.18974	66.2682	75.6393	1.06384	67.372	76.8992	1.08156	73.435	83.8195	1.1789	54.9249	62.6919	0.88174
oh et al. 20	WR20	CB	0.03596	56.0735	69.5124	0.85924	46.9031	58.1442	0.71872	44.6776	55.3853	0.68461	51.1928	63.4619	0.78445	43.1848	53.5347	0.66174
	WR10	CB	0.03818	75.1728	93.3783	1.08706	62.7405	77.9351	0.90728	59.5995	74.0334	0.86186	68.6437	85.2679	0.99264	57.8809	71.8986	0.837
	WB	CB	0.03764	67.8167	83.2335	0.98269	56.5278	69.3783	0.81911	53.6756	65.8778	0.77778	61.888	75.9571	0.89678	52.0174	63.8426	0.75375

Ref.	Specimen	failure type	drift	l_{p1}			l_{p2}			l_{p3}			l_{p4}			l_{p5}		
				Δ_f	Δ_u	cal/exp	Δ_f	Δ_u	cal/exp	Δ_f	Δ_u	cal/exp	Δ_f	Δ_u	cal/exp	Δ_f	Δ_u	cal/exp
deng 2008	HPCW01	CB	0.02	31.0954	34.7606	0.82763	31.9191	35.6814	0.84956	46.8173	52.3357	1.24609	48.1081	53.7785	1.28044	34.5793	38.6551	0.92036
	HPCW02	BR	0.0248	36.4509	41.0883	0.78895	37.4501	42.2147	0.81057	55.522	62.5858	1.20172	57.0877	64.3507	1.23561	40.677	45.852	0.88042
	HPCW03	CB	0.02448	44.4666	50.1863	0.97639	45.7243	51.6058	1.004	68.471	77.2784	1.50347	70.4417	79.5026	1.54674	49.7858	56.1898	1.09319
	HPCW04	BR	0.02677	50.0164	56.9868	1.01364	51.4602	58.6319	1.0429	77.5725	88.3833	1.5721	79.8348	90.9608	1.61794	56.1226	63.9441	1.13739
thomsen	RW1	BR	0.0226	98.8242	108.634	1.31333	103.11	113.345	1.37029	146.87	161.449	1.95185	143.43	157.667	1.90612	121.608	133.678	1.61611
	RW2	CB	0.0235	111.332	123.516	1.43607	116.287	129.013	1.49998	166.873	185.137	2.1525	162.896	180.724	2.1012	137.669	152.737	1.7758
segura 2007	WP1	CB	0.02451	252.48	271.865	1.29581	254.151	273.663	1.30438	349.472	376.304	1.7936	320.317	344.91	1.64396	314.894	339.071	1.61613
	WP2	CB	0.02549	406.916	438.675	2.01056	409.939	441.935	2.0255	582.516	627.981	2.87819	529.731	571.076	2.61738	519.913	560.492	2.56887
	WP3	CB	0.01982	364.024	391.95	2.31005	366.668	394.796	2.32682	517.54	557.243	3.28424	471.394	507.556	2.9914	462.811	498.315	2.93693
	WP4	0	0.0388	1104.44	1177.34	2.26438	921.805	982.65	1.88994	1361.83	1451.72	2.79211	993.257	1058.82	2.03643	1430.81	1525.25	2.93352
	WP5	CB	0.0282	566.343	612.578	2.65901	561.857	607.726	2.63795	798.688	863.892	3.74989	749.2	810.364	3.51754	718.809	777.492	3.37485
	WP6	CB	0.03196	527.592	571.73	2.1896	523.455	567.248	2.17243	741.857	803.921	3.07884	696.22	754.466	2.88943	668.194	724.095	2.77312
	WP7	CB	0.0397	561.753	610.19	1.88122	557.312	605.366	1.86635	791.787	860.059	2.65157	742.791	806.838	2.48749	712.702	774.155	2.38673
lowes 2011	PW1	R0	0.0153	306.088	334.732	2.52574	345.306	377.62	2.84935	381.366	417.055	3.1469	492.486	538.574	4.06383	388.249	424.582	3.2037
	PW2	CB	0.015	126.183	142.954	1.56233	159.925	181.181	1.98012	179.549	203.413	2.22309	247.626	280.539	3.066	153.673	174.098	1.90271
	PW3	CB	0.0122	101.672	112.807	1.51582	137.716	152.799	2.0532	174.895	194.05	2.6075	210.703	233.779	3.14135	127.627	141.605	1.90279
	PW4	CB	0.0101	146.513	166.182	2.69731	195.356	221.581	3.59651	254.064	288.171	4.67733	306.116	347.211	5.63562	183.635	208.287	3.38073
oesterle 1976 -1979	R1	BR	0.0226	166.015	185.576	1.79601	198.654	222.062	2.14911	276.121	308.657	2.98718	302.835	338.518	3.27618	208.855	233.465	2.25947
	R2	BR	0.0292	170.107	189.847	1.42205	197.127	220.002	1.64792	279.63	312.079	2.33763	301.249	336.207	2.51836	210.764	235.222	1.76193
	B1	BR	0.0289	168.832	196.906	1.49024	195.66	228.196	1.72705	277.576	323.733	2.45009	299.042	348.769	2.63957	209.2	243.987	1.84656
	B3	BR	0.0393	168.795	197.065	1.09676	196.87	229.842	1.27918	278.447	325.082	1.80923	301.022	351.439	1.95592	209.867	245.016	1.36363
	B4	R0	0.0594	43.2196	50.4582	0.1858	48.4744	56.5931	0.20839	64.5189	75.3247	0.27736	68.7234	80.2334	0.29544	51.1264	59.6892	0.21979
Liu 2005	W1	CB	0.02939	210.193	226.295	2.05349	175.845	189.316	1.71793	255.331	274.89	2.49446	245.841	264.674	2.40176	237.727	255.938	2.32249
	W2	BR	0.0288	169.709	183.697	1.7009	142.534	154.282	1.42854	205.42	222.352	2.05882	197.912	214.226	1.98357	191.493	207.277	1.91923
Chen 2005	W3	CB	0.03795	239.641	261.832	1.84	199.982	218.501	1.5355	291.756	318.774	2.24015	280.8	306.803	2.15603	271.431	296.567	2.08409
	W4	CB	0.03064	239.673	263.529	2.29355	200.182	220.107	1.91564	291.569	320.59	2.79016	280.658	308.593	2.68576	271.33	298.336	2.59648

Ref.	Specimen	failure type	drift	l _{p6}			l _{p7}			l _{p8}			l _{p9}			l _{p10}		
				Δ _f	Δ _u	cal/exp	Δ _f	Δ _u	cal/exp	Δ _f	Δ _u	cal/exp	Δ _f	Δ _u	cal/exp	Δ _f	Δ _u	cal/exp
Deng 2008	HPCW01	CB	0.02	34.9975	39.1226	0.93149	32.6205	36.4655	0.86823	32.068	35.8478	0.85352	36.2858	40.5628	0.96578	28.4103	31.759	0.75617
	HPCW02	BR	0.0248	41.1843	46.4239	0.8914	38.3685	43.2499	0.83045	37.6307	42.4182	0.81448	42.8424	48.2929	0.92728	33.6775	37.962	0.72892
	HPCW03	CB	0.02448	50.4244	56.9105	1.10721	46.8903	52.9218	1.02961	45.9516	51.8623	1.00899	52.5256	59.2819	1.15334	41.0198	46.2961	0.9007
	HPCW04	BR	0.02677	56.8557	64.7793	1.15225	52.8587	60.2253	1.07124	51.7211	58.9291	1.04819	59.3524	67.6239	1.20284	46.5006	52.9811	0.94239
thomsen	RW1	BR	0.0226	102.593	112.776	1.36342	101.861	111.972	1.35369	109.784	120.681	1.45898	115.81	127.305	1.53907	76.9841	84.6256	1.02309
	RW2	CB	0.0235	115.688	128.35	1.49227	116.669	129.438	1.50492	124.001	137.572	1.59949	133.553	148.17	1.72271	89.3837	99.1662	1.15296
segura 2007	WP1	CB	0.02451	236.708	254.882	1.21486	251.858	271.195	1.29261	277.868	299.202	1.4261	287.253	309.308	1.47427	187.391	201.779	0.96175
	WP2	CB	0.02549	378.361	407.891	1.86947	409.268	441.211	2.02218	452.88	488.227	2.23766	474.796	511.853	2.34595	292.064	314.86	1.44308
	WP3	CB	0.01982	339.061	365.071	2.15163	366.35	394.454	2.3248	404.208	435.216	2.56504	423.748	456.256	2.68905	262.961	283.134	1.66871
	WP4	0	0.0388	701.18	747.462	1.4376	925.495	986.584	1.8975	1102.6	1175.38	2.26062	1121.35	1195.37	2.29906	542.106	577.889	1.11146
	WP5	CB	0.0282	525.124	567.994	2.46549	564.755	610.861	2.65156	619.278	669.835	2.90755	659.223	713.042	3.09509	402.163	434.995	1.88818
	WP6	CB	0.03196	489.58	530.539	2.03184	525.279	569.224	2.18	576.408	624.631	2.39219	612.043	663.247	2.54009	373.396	404.634	1.54966
	WP7	CB	0.0397	520.944	565.863	1.74456	560.502	608.831	1.87703	614.162	667.118	2.05673	654.163	710.568	2.19069	398.193	432.527	1.33349
lowes 2011	PW1	R0	0.0153	349.623	382.341	2.88497	342.457	374.505	2.82584	364.89	399.037	3.01095	389.422	425.864	3.21338	261.022	285.448	2.15386
	PW2	CB	0.015	178.198	201.883	2.20637	156.621	177.437	1.93921	159.5	180.699	1.97486	172.403	195.317	2.13461	131.396	148.86	1.62689
	PW3	CB	0.0122	152.924	169.672	2.27993	136.592	151.552	2.03645	137.363	152.407	2.04793	150.402	166.875	2.24234	116.152	128.873	1.7317
	PW4	CB	0.0101	218.434	247.757	4.02138	192.256	218.065	3.53944	194.819	220.972	3.58663	212.634	241.179	3.9146	158.99	180.334	2.92702
oesterle 1976 - 1979	R1	BR	0.0226	212.287	237.301	2.29659	206.719	231.077	2.23636	204.372	228.454	2.21097	236.255	264.094	2.5559	172.64	192.982	1.86768
	R2	BR	0.0292	210.752	235.208	1.76183	205.187	228.997	1.71531	202.842	226.38	1.6957	234.707	261.943	1.96209	170.35	190.118	1.42408
	B1	BR	0.0289	209.188	243.974	1.84646	203.663	237.53	1.79769	201.335	234.814	1.77714	232.973	271.714	2.0564	170.554	198.915	1.50544
	B3	BR	0.0393	210.499	245.754	1.36773	204.932	239.255	1.33157	202.587	236.516	1.31632	234.461	273.729	1.52343	170.932	199.561	1.11065
	B4	R0	0.0594	51.1241	59.6866	0.21978	50.0419	58.4231	0.21513	49.5859	57.8907	0.21317	55.7828	65.1255	0.2398	43.3031	50.5556	0.18616
Liu 2005	W1	CB	0.02939	172.513	185.728	1.68537	176.31	189.816	1.72247	189.303	203.805	1.84941	203.458	219.043	1.98769	129.623	139.553	1.26636
	W2	BR	0.0288	139.897	151.429	1.40212	146.197	158.248	1.46525	153.181	165.808	1.53526	169.038	182.971	1.69418	112.567	121.846	1.1282
Chen 2005	W3	CB	0.03795	196.135	214.298	1.50596	199.018	217.448	1.52809	215.521	235.479	1.65481	229.74	251.014	1.76398	143.674	156.978	1.10315
	W4	CB	0.03064	196.351	215.895	1.87898	205.418	225.864	1.96574	215.655	237.12	2.06371	238.574	262.32	2.28303	156.456	172.029	1.4972



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