Coupling Beams of Shear Walls
Modelling Procedure for the Seismic Analysis of RC Structures

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ABSTRACT: Reinforced concrete (RC) shear walls with couplings beams are used in many buildings as primary lateral load-bearing elements. This type of beams has special characteristics, such as large deformation demands and a shear-dominated response. An accurate seismic assessment of such structural system is decisively influenced by the model adopted for those discontinuity (D) regions. Currently, many technical and normative seismic design documents do not take into account the distinctive behavior of coupling beams.

The aim of this work is the study and development, based on a critical and comparative analysis of the existing literature, of a modelling procedure for the behavior of RC coupling beams, to be used in the seismic design/assessment of structural systems. To achieve this goal, (i) a modelling procedure is proposed, based on the experimental studies by Breña & Ihtiyar (2007), and (ii) the seismic performance of a RC “coupled shear walls – frame” system is evaluated, in which the proposed procedure is used.

It is shown (i) that the effective stiffness suggested in the current design documents is in excess when applied to coupling beams, (ii) that the methods recommended in Eurocode 2 (EC2) and Eurocode 8 (EC8) are adequate to coupling beams without shear problems, and (iii) that the proposed modelling procedure, by taking into account shear behavior and shear failure, is mainly relevant in the seismic performance of structures that possess coupling beams with inadequate transverse reinforcement. This is the case of several RC existing old buildings.

KEYWORDS: Reinforced Concrete (RC) Structures, Seismic Performance, Nonlinear Analysis, Modelling Procedure, D Regions, Coupling Beams

1. Introduction

Traditionally, the seismic design and assessment of reinforced concrete (RC) structures have been based on strength criteria. More recently, new performance-based methods have been proposed, relying on nonlinear static (pushover) analysis.

The seismic performance of RC structures is very influenced by the behavior of the critical, or discontinuity (D), regions. These are the regions where the hysteretic energy dissipation mainly occurs, and are subject to large deformation demands (EN 1998-1, 2004). Coupling beams between structural walls are an example of critical regions.

This paper studies a modelling procedure, defined based on the existing literature, aiming to represent the behavior of RC coupling beams (with conventional longitudinal reinforcement), and to be used in nonlinear static analyses.
2. Coupling Beams

Stairway and lifts cores are preferential locations for shear walls, in many medium and tall buildings. Since the access to the stairs and elevators requires openings in the walls, they may be disconnected from each other, or connected by small wall segments between each opening – coupling beams.

These may be defined as “beams that are used to connect adjacent concrete wall elements to make them act together as a unit to resist lateral loads” (ASCE/SEI, 2010), and are represented in Figure 1.

When the coupled wall system undergoes lateral displacements, the coupling beams are subjected to high deformation demands, playing a key role in the overall seismic response of the building.

2.1. Characteristics and benefits

The coupling effect presents three main benefits (El-Tawil et al., 2010):

(i) The binary effect generated by the accumulated shear resistances of coupling beams contributes to the total resisting base moment, which reduces the bending moments that would need to be resisted by each wall if they acted individually.

(ii) It provides a means of dissipating seismic energy, as the coupling beams undergo inelastic deformations.

(iii) It significantly increases the lateral stiffness of the system, when compared to uncoupled walls.

For these abovementioned benefits to be achieved, coupling beams need to be designed and detailed in a way that enables them to have a ductile behavior, maintaining their strength and energy dissipation capacity while undergoing large cyclical deformations. Because coupling beams are typically short and deep elements, with a low span-to-depth ratio, their inelastic behavior is influenced by high shear forces.

In order to minimize strength and stiffness deterioration during earthquake loading, and to avoid brittle shear failure, some studies suggest the use of diagonal reinforcement, which experimentally has been shown to exhibit a good performance (ASCE/SEI, 2013). Besides diagonal reinforcement other alternatives been developed, such as the introduction of a steel shear plate, hybrid systems (concrete walls with steel coupling beams), among others.

Although many alternatives exist, this study focus on conventionally reinforced coupling beams (with longitudinal reinforcement for flexure and transverse reinforcement for shear), which are common in most older RC buildings (ASCE/SEI, 2013).
3. Modelling Procedure

The modelling procedure studied is based on experimental results by Breña & Ihtiyar (2007).

3.1. Experimental Study (Breña & Ihtiyar, 2007)

Four conventionally reinforced concrete coupling beams were designed to develop different kinds of behavior, by varying the three key parameters that control the failure mode (span length to depth ratio, amount of longitudinal reinforcement, and amount of transverse reinforcement), and subsequently tested. The main specimen geometry and reinforcing details are represented in Figure 2 and indicated in Table 1, together with material properties (Breña et al., 2009).

![Figure 2 – Specimen geometry and reinforcing details, measures in meters (Breña et al., 2009)](image)

Table 1 – Specimen geometry, reinforcing details and material properties (Breña et al., 2009)

<table>
<thead>
<tr>
<th>Specimen</th>
<th>(d) (m)</th>
<th>(l_b) (m)</th>
<th>(l_b/h)</th>
<th>Longitudinal reinforcement</th>
<th>Transverse reinforcement</th>
<th>(f_c) (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB-1</td>
<td>0.34</td>
<td>0.51</td>
<td>1.3</td>
<td>600 517</td>
<td>0.69 142 524</td>
<td>39</td>
</tr>
<tr>
<td>CB-2</td>
<td>0.34</td>
<td>1.02</td>
<td>2.7</td>
<td>851 448</td>
<td>0.99 52 607</td>
<td>1.13 39</td>
</tr>
<tr>
<td>CB-3</td>
<td>0.27</td>
<td>0.51</td>
<td>1.3</td>
<td>860 517</td>
<td>1.25 142 524</td>
<td>1.1 31</td>
</tr>
<tr>
<td>CB-4</td>
<td>0.34</td>
<td>1.02</td>
<td>2.7</td>
<td>400 517</td>
<td>0.47 142 524</td>
<td>1.1 30</td>
</tr>
</tbody>
</table>

In Table 1 \(d\) represents the flexural depth, \(l_b\) the coupling beam span, \(l_b/h\) the span to depth ratio, \(A_s\) the reinforcement area, \(f_y\) the steel reinforcement yield stress, \(\rho_s\) the steel reinforcement ratio and \(f_c\) the concrete compressive strength. The subscripts \(l\) and \(w\) represent the longitudinal and transverse reinforcements, respectively.

The test setup is shown in Figure 3.

![Figure 3 – Experimental setup (Breña et al., 2009)](image)

The coupling beams connected two RC walls with a height \(h_w\) of 1.40m between the supporting pins, which allowed free rotation at both ends of the walls. Lateral loading was imposed at the top of the walls through a rigid loading beam that imposed equal displacement to both walls.

3.2. Numerical Modelling

The experiment is reproduced using the structural analysis software SAP2000 (CSI, 2014). All elements are modelled by beam elements, and the inelastic behavior of the coupling beams is simulated with plastic hinges. As it is intended that the modelling procedure is easily applied, to be used in design offices, the simplicity of lumped plasticity models (implicitly including all aspects of the element behavior) determines the choice.

In SAP2000 the linear behavior is modeled by the frame elements, and subsequent deformation beyond the elastic limit occurs within the plastic hinges. Two moment hinges are placed at both ends of the coupling beams and one shear hinge at midspan.
3.3. Linear Behavior

In order to adequately model the real behavior of the coupling beams characterized by the linear phase, an effective stiffness is defined to take into account the nonlinear effects that occur before yielding (e.g. to represent the cracking and other deformation components).

Concrete Modulus of Elasticity

As the value was not defined experimentally, it has to be estimated based on the concrete compressive strength. Several expressions, based on different design codes, are used, resulting in similar values – Table 2.

<table>
<thead>
<tr>
<th>Beam</th>
<th>$f_{cm}$ (MPa)</th>
<th>NZS 3101</th>
<th>CSA A23.3</th>
<th>ACI 319-11</th>
<th>EC 2</th>
<th>Chosen Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>CB-1</td>
<td>39</td>
<td>30.1</td>
<td>31.5</td>
<td>31.2</td>
<td>33.1</td>
<td>31</td>
</tr>
<tr>
<td>CB-2</td>
<td>39</td>
<td>30.1</td>
<td>31.5</td>
<td>31.2</td>
<td>33.1</td>
<td>31</td>
</tr>
<tr>
<td>CB-3</td>
<td>31</td>
<td>28.2</td>
<td>28.8</td>
<td>28.2</td>
<td>30.9</td>
<td>29</td>
</tr>
<tr>
<td>CB-4</td>
<td>30</td>
<td>27.9</td>
<td>28.5</td>
<td>27.8</td>
<td>30.6</td>
<td>29</td>
</tr>
</tbody>
</table>

Effective Stiffness

The effective stiffness $E_c I_e$ of an element may be represented by a dimensionless factor, $\kappa$, which represents it as a fraction of the gross elastic stiffness $(E_c I_g)$:

$$\kappa = \frac{E_c I_e}{E_c I_g} \quad (1)$$

Where $I_g$ and $I_e$ represent the gross and effective moments of inertia, respectively. In Figure 4 is plotted $\kappa$ versus span to depth ratio for several effective stiffness formulations. As it is shown, there is a high dispersion of the results obtained from the different expressions, for the same span to depth ratio.

Most of the current design documents do not specify an effective stiffness value for coupling beams.

The value prescribed is usually constant and does not depend on important parameters that influence the behavior of coupling beams, such as the span to depth ratio, the reinforcement content, or the type of detailing. For this reason, important deformation components are ignored or underestimated in such cases.

Usually the deformation parameter used to describe the behavior of coupling beams is the chord rotation, $\theta_c$, which can be decomposed in three main components: flexure ($\theta_f$), shear ($\theta_v$) and longitudinal bar slippage ($\theta_s$). In regular beams the flexure rotation is dominant, but in short and deep beams, such as coupling beams, the remaining component play an important role in post-cracking, pre-yield deformations, even for small displacement demands (Breña et al., 2009). The formulation by Son Vu et al. (2014) was developed specifically to coupling beams, taking into account shear and bar slippage deformations.
### 3.4. Nonlinear Behavior

The nonlinear behavior is defined through force-deformation relations. First the yield and ultimate strengths are determined, and then the deformation parameters and residual strength are obtained through ASCE 41-13 – Figure 5.

![Deformation and Deformation ratio](image)

**Figure 5 – Force-Deformation Relations (ASCE/SEI, 2013)**

### Yield and Ultimate Strength

The yield and ultimate flexural strengths are determined through a moment-curvature analysis of the coupling beams sections.

The steel stress-strain relationship is defined by Mander’s (1983) model, unconfined concrete by the EC 2 model (NP EN 1992-1-1, 2010), and confined concrete by the EC8-2 Annex E model (EN 1998-2, 2005). The beam’s cross sections are modelled in SAP2000’s Section Designer, and the yield \(M_y\) and ultimate \(M_u\) flexural strengths are obtained through moment-curvature analyses. Shear strength \(V_R\) is determined through the formulations proposed in ACI 318-11 and EC 2.

### Deformation Parameters

The deformation parameters “a”, “b”, “d” and “e”, as well as the residual strength ratio “c” (Figure 5), are obtained from ASCE 41-13 prescriptions for conventionally reinforced coupling beams.

Since the SAP2000 hinge curves only account for post-yield deformations, it is necessary to subtract the yield chord rotation from the values “d” and “e”. It is assumed a simplified linear elastic model where all deformation components pre-yielding are included in the effective stiffness value – Figure 6.

![Elastic coupling beam model](image)

**Figure 6 – Elastic coupling beam model**

The chord rotation when the shear strength is reached \(\theta_{c,R}\) is given by:

\[
\theta_{c,R} = \frac{V_R}{12E_cI_c}t_b^2
\]  

(2)

Furthermore, in ASCE 41-13, “d” and “e” values refer to chord rotations, while in SAP2000 the shear hinge curves are defined in displacement terms. Thus, it is needed to perform the necessary transformations:

\[
d' = (d - \theta_{c,k}) \cdot t_b
\]  

(3)

\[
e' = (e - \theta_{c,k}) \cdot t_b
\]  

(4)

### 3.5. Numerical Models

In Table 3 is the depicted the summary of the different numerical models for the coupling beams: CB-1, CB-2, CB-3 and CB-4.

<table>
<thead>
<tr>
<th>CB-1</th>
<th>CB-2</th>
<th>CB-3</th>
<th>CB-4</th>
</tr>
</thead>
<tbody>
<tr>
<td>(l_b/h = 1.3)</td>
<td>(l_b/h = 2.7)</td>
<td>(l_b/h = 1.3)</td>
<td>(l_b/h = 2.7)</td>
</tr>
<tr>
<td>(E_c = 31) GPa</td>
<td>(E_c = 31) GPa</td>
<td>(E_c = 29) GPa</td>
<td>(E_c = 29) GPa</td>
</tr>
<tr>
<td>(\kappa = 0.05) (Son Vu et al., 2014)</td>
<td>(\kappa = 0.10) (Son Vu et al., 2014)</td>
<td>(\kappa = 0.09) (Son Vu et al., 2014)</td>
<td>(\kappa = 0.12) (Son Vu et al., 2014)</td>
</tr>
<tr>
<td>(M_y = 97) kNm</td>
<td>(M_y = 117) kNm</td>
<td>(M_y = 113) kNm</td>
<td>(M_y = 65) kNm</td>
</tr>
<tr>
<td>(M_u = 113) kNm</td>
<td>(M_u = 130) kNm</td>
<td>(M_u = 151) kNm</td>
<td>(M_u = 75) kNm</td>
</tr>
<tr>
<td>(V_y = 492) kN</td>
<td>(V_y = 187) kN</td>
<td>(V_y = 439) kN</td>
<td>(V_y = 432) kN</td>
</tr>
<tr>
<td>Flexure:</td>
<td>Flexure:</td>
<td>Flexure:</td>
<td>Flexure:</td>
</tr>
<tr>
<td>Shear:</td>
<td>Shear:</td>
<td>Shear:</td>
<td>Shear:</td>
</tr>
<tr>
<td>Hinge:</td>
<td>Hinge:</td>
<td>Hinge:</td>
<td>Hinge:</td>
</tr>
<tr>
<td>(a = 0.020)</td>
<td>(a = 0.013)</td>
<td>(a = 0.010)</td>
<td>(a = 0.024)</td>
</tr>
<tr>
<td>(b = 0.040)</td>
<td>(b = 0.028)</td>
<td>(b = 0.025)</td>
<td>(b = 0.049)</td>
</tr>
<tr>
<td>(c = 0.500)</td>
<td>(c = 0.320)</td>
<td>(c = 0.250)</td>
<td>(c = 0.717)</td>
</tr>
</tbody>
</table>

Table 3 – Summary of the numerical models
3.6. Modelling Procedure Outline

In Figure 7 is presented the proposed modelling procedure outline.

### 1. Elastic Properties (Concrete)

- **EC 2 (NP EN 1992-1-1, 2010)**
  
  \[ E_{cu} = 22 \left( \frac{f_{ck}}{40} \right)^3 \]

### 2. Effective Stiffness – Son Vu et al. (2014)

\[ k_{eff} = 0.67 \left(1.0 + 0.25 f_{ctk} \right) \left(0.009 + 0.7 f_{yf} + 1.5 f_{yf} \right) \left(0.5 + \frac{f_{ctk}}{f_{ck}} \right) \]

### 2. Yield and Ultimate Strengths

#### a) Flexure – Moment-Curvature Analysis

- Steel – Pippa (1993) and Mander (1983)
- Concrete (unconfined) – EC 2 (NP EN 1992-1-1, 2010)

#### b) Shear – ACI 318-11 or EC 2

- **EC 2 (NP EN 1992-1-1, 2010)**
  
  \[ V_u = \frac{4}{3} f_{ck} d \text{ (con} \) \left( \sigma_{cr} \right) \]

- **ACI 318-11 (ACI, 2011)**
  
  \[ V_u = 0.8 f_{yf} d \left[ 1 + 0.5 \left( \frac{f_{ctk}}{f_{ck}} \right) \right] \]

#### iii) \( M_S, M_L \) – SAP2000 Section Designer

### 3. Deformation Parameters

- **Flexure and Shear – ASCE 41-13 (ASCE/SEI, 2013)**

#### Force-Deformation Relations (ASCE 41-13)

#### Curves adapted to coupling beams

#### SAP2000 Hinges (post-yield behavior)

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**Figure 7** – Outline of the proposed modelling procedure for conventionally reinforced concrete coupling beams

3.7. Results and Analysis

In Figure 8 are represented the experimental results, as well as the curves obtained from three types of numerical models, which differ only on the formulation used for the coupling beams’ effective stiffness: (i) Son Vu et al. (2014); (ii) FEMA 356, ASCE 41-06 or EC 8; (iii) experimental effective stiffness, based on the experimental tests. To make the comparison easier to understand and clearer, only the results concerning one direction are presented.

As it is shown, the value chosen to model the effective stiffness is very important for a good adjustment to the experimental results. The effective stiffness dictates the chord rotation value at longitudinal reinforcement yielding or at shear failure.

The experimental effective stiffness values are extremely low (1-15%) when compared to the values recommended in several design codes (30-50%). The formulation by Son Vu et al. (2014) produces the closest effective stiffness values to experimental results. In beams CB-2 and CB-4 the numerical approximation is almost exact. In beams CB-1 and CB-3, although the numerical and experimental effective stiffness values are close in absolute terms (2% vs 5%, respectively), since they are very low they result in fairly different yield rotations.
The different behavior and failure modes are adequately represented by the numerical models. The model of the beam CB-2 is shear controlled, which correctly matches the experimental results, were the beam failed suddenly to insufficient shear capacity. The models for beams CB-1 and CB-4 are controlled by flexure, and the model for beam CB-3 also starts being controlled by flexure, but reaches the shear strength before the ultimate moment.

In general the deformation parameters (chord rotations) in the models show a very good fit with the experimental results. This is very important for an adequate performance-based seismic assessment and design. The yield moments also demonstrate a very good correspondence. The calculated flexural and shear strengths result in a fair approximation of the experimental results, and the differences are all on the conservative side.

The overall response of the coupling beams tested by Breña & Ihtiyar (2007) is considered to be adequately modelled by the proposed procedure.

4. Case Study

To examine the influence of the proposed modelling procedure on the overall seismic performance of a RC building, a coupled wall–frame structural system representative of a “common” building is designed and studied – Figure 9.

The building is designed based on EC 2 and EC 8. The seismic action is considered through a design spectrum for linear analysis, which includes a behavior factor $q$ to account for the nonlinear behavior of structural elements. The adopted dimensions, material properties and design procedure are described in Bezelga (2015).
4.1. Nonlinear Static Analysis

The seismic performance of the system is then assessed through the N2 Method (Fajfar, 2000), which is based on a nonlinear static (pushover) analysis. Lumped plasticity models are used to model the nonlinear behavior of the structural elements. For the coupling beams the proposed modelling procedure is used. For the remaining elements the nonlinear modelling procedure suggested in EC 8 is followed: a global effective stiffness value of 0.50 and the respective moment-curvature relations are used, along with a hinge length value throughout which the curves are integrated (Bezelga, 2015).

In Figure 10 is represented the acceleration-displacement response spectra (ADRS) for two types of earthquakes (SA 1.3, SA 2.3) and the capacity curve of one equivalent single degree of freedom (SDOF) system, with the target and ultimate displacements (marked with a circle and a square, respectively).

The damage states for the target and ultimate displacements of the coupled wall-frame system are presented in Figure 11.

Since the target displacement is very close to the elastic limit, the ductility and nonlinear behavior of the system are almost not explored for the design seismic action. The collapse of the system is by flexural failure of one of the coupling beams. This type of failure is expected as the system was designed by capacity design.

4.2. Influence of the Effective Stiffness of Coupling Beams

To better understand the influence of the effective stiffness of coupling beams on the seismic assessment of RC structures, a second model is considered where the value chosen is the one prescribed in EC 8, and which was used in the previous analyses for the other elements (κ = 0,50). It is considered a seismic intensity three times higher than the design seismic action to increase the structure sensitivity to the modelling differences – Figure 12.

It is shown that the use of a higher effective stiffness for coupling beams results in a less conservative response, as the target displacement is more distant from the ultimate displacement when compared to the use of a low effective stiffness. The higher the seismic intensity is, the stronger this effect is. (Bezelga, 2015)
4.3. Influence of the Nonlinear Model of Coupling Beams

In all previous analyses the proposed modelling procedure was used to represent the nonlinear behavior of coupling beams. A new analysis is done in which the coupling beam nonlinearity is modelled like suggested in EC 8 and like it has been modelled in the remaining elements: through the definition of the moment-curvature relation and of an effective hinge length.

The moment-curvature curve is obtained from SAP2000’s Section Designer. Two hinge lengths are used: 0.50m - half of the elements’ cross section height (Paulay & Priestley, 1992), and 0.30m - based on EC 8 (EN 1998-2, 2005).

In Figure 13 are shown the obtained results.

![Figure 13 – ADRS with influence of the nonlinear model adopted for coupling beams](image)

Because the only difference between the models is the plastic deformation capacity considered, the elastic behavior of all models is equal. However, the use of a 0.50m hinge length results in a system with a 50% higher ultimate displacement capacity when compared to the use of a 0.30m value. Thus, the most conservative model for the system studied is the one based on EC 8 plastic hinge length, followed by the use of ASCE 41-13 parameters, with a 16% difference.

4.4. Influence of Shear Failure in Coupling Beams

Since it was followed the capacity design procedure for the design of the coupled wall-frame system, in all previous analyses the collapse was due to the flexural failure of the structural elements (coupling beams in most cases). Thus, to study the influence of shear failure, the shear reinforcement of the coupling beams is defined based on the shear forces demands, reducing from Φ10/0.15 to Φ10/0.20. This change was enough to significantly change the behavior of the whole system – Figure 14.

![Figure 14 – ADRS with influence of shear failure in coupling beams](image)

As it is shown, when using the proposed modelling procedure (in blue) both maximum strength and ultimate displacement capacity are greatly reduced (15% and 40% respectively). This happens because the collapse is now being controlled by shear failure in one of the coupling beams. The other nonlinear models are not affected by this change, as they do not take into account shear deformations/failure.

5. Conclusion

A modelling procedure to represent the behavior of conventionally reinforced coupling beams was developed and studied.

Linear Behavior

Most of the current codes and design documents prescribe a global effective stiffness value of 50% to account for the cracking of structural elements during seismic analysis. Based on the study developed it is considered that this value is not appropriate to represent the particularities of the hysteretic behavior of
couplings beams, as (i) it does not account for important specific deformation components, such as shear and longitudinal bar slippage deformations, and (ii) its use resulted in a higher deformation capacity (not conservative) for the coupled wall-frame system analyzed, when compared with the use a lower effective stiffness value for the coupling beams. The formulation by Son Vu et al. (2014) is favored as it was developed specifically for couplings beams and lead to a good fit with the studied experimental results.

**Nonlinear Behavior**

The nonlinear behavior is defined by force-deformation relationships. The methods to determine the yield and ultimate strengths are generally accepted and lead to similar results. On the other hand, the way to model the deformation capacities of coupling beams is less explored. Although the method suggested in EC 8 resulted in a conservative response of the coupled wall-frame system studied, when compared with using the ASCE 41-13 parameters, it is highly influenced by the value chosen for plastic hinge length. Additionally, the method in EC 8 is not specifically developed for coupling beams, not taking into account shear deformations and consequent failure.

The use of the deformation parameters suggested in ASCE 41-13, defined both for shear and flexure controlled coupling beams, resulted in a good correlation with the type of failure and ductility associated with the tests of the coupling beams by Breña & Ihtiyar (2007). Additionally, and very important, when studying the coupled wall-frame system the ASCE 41-13 models were the only ones able to represent shear failure in coupling beams, which significantly compromises the system response. These conclusions emphasize the importance of the proposed procedure mainly for two types of use: (i) in the design of new buildings, to more adequately account for the contribution of coupling beams in the horizontal force resisting system, and mainly (ii) in the seismic assessment and rehabilitation of old RC buildings, which are specially prone to exhibit inadequate transverse reinforcement and thus are susceptible to brittle shear failure of the coupling beams.

**References**


