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APPROACHING THE CSA S304-14 DUCTILITY VERIFICATION FROM A SOFTWARE DESIGN PERSPECTIVE

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ABSTRACT

In the process of updating the masonry design software Masonry Analysis Structural Systems (MASS) to include the added scope of the seismic design in Clause 16 of CSA S304-14, several technical issues arose related to performing and satisfying the ductility verification for moderately ductile and ductile shear walls. To deal with the prohibitively long calculation times associated with repeating ductility verification attempts for each failing cross section, a methodology was developed to allow the software to design shear walls that satisfy the ductility verification. For cases where increasing compressive strain is not an available option for a shear wall cross section, the software determines a target neutral axis depth to compare for future design iterations. Following this, MASS increments cross sectional properties and compares the neutral axis depth to reach the saved target value. Alternately, a failure message is displayed if the target value can not be reached within the user defined cross-section parameters. In the case of shear walls containing boundary elements that initially fail a ductility verification attempt, it is possible for the maximum compressive strain to be increased to improve ductility without changing cross sectional geometric or material properties. The software first determines the required increase in compressive strain that must be achieved, before comparing that value to other potential limiting factors. These factors include code minimums and maximums, strain within the shear wall web, confinement from ties within the boundary element, and the interaction of any strain increase with neutral axis depth. This methodology is valuable for addressing ductility verification failures by reducing the number of ductility verification iterations from the tens of thousands down to single digits, making substantial improvements in calculation times and more easily allowing engineers to find workable designs. Additional recommendations are made for designers to consider that fall beyond the scope of the software.

KEYWORDS: *seismic design, software, ductility verification, shear walls*

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INTRODUCTION

Masonry shear walls can be subjected to a requirement that they must satisfy a ductility verification calculation specified in CSA S304: 16.8.8[1]. This is based on the desired ductility if the seismic force resisting system (SFRS) is comprised of masonry shear walls with a non-squat height-to-length aspect ratio and classified as either *moderately ductile* or *ductile*.

Engineers may wish to take advantage of a higher ductility SFRS to reduce the design earthquake loads or may be forced to consider increased ductility for *post-disaster* importance category structures as it is a building code requirement in NBCC 2015 4.1.8.10(2)[2]. In either scenario, satisfying requirements by ensuring adequate ductility performance can often prove to be a significant hurdle in the design process for engineers to overcome.

While the discussed approach and design process mainly describe how the masonry design software Masonry Analysis Structural Systems (MASS) executes the ductility verification, the approach can be mimicked by engineers through hand calculations, in their own spreadsheets, or in-house programming. Additional recommendations are made at the end of this paper to clarify the limits of the software's approach and suggestions are given for engineers to find a workable design solution that are beyond the scope of MASS.

SOFTWARE DESIGN PROCESS REVIEW

Before any consideration of ductility was within the scope of the software, design had been limited to a series of tests where either a failure is flagged, and design is stopped, or success permits the next test to be run. The software only generates a successful design result if the designed assemblage satisfies all requirements of CSA S304[1]. For example, the software evaluates S304: 10.15.1.1[1] by comparing the section's reinforcement to the minimum area required. This is but one test in a long series of design considerations that are performed before a design has been iterated to the point of having passed all the necessary checks.

The MASS software is intentionally designed to not execute any design decisions requiring critical engineering judgement. Instead, it simply runs through a predefined sequence of tests before arriving at a result and displays the success or failure for the last section attempted.

A need for change

The ductility verification was initially planned to be executed as similar design criteria where it is checked, and a failure is either reported or the design is successful and marked as complete; however, this was not possible for the reasons described below.

Firstly, there are several different options to potentially address a failed ductility verification and the immediate response of adding reinforcement is less likely to address ductility failure in the same way it would for moment or shear design. Furthermore, ductility verifications in multi-storey shear wall designs are typically more difficult to satisfy due to the higher axial loads present on the critical section. In addition, increasing section properties for the base element may be ineffective if inelastic rotational demand experienced at the base where it is checked is driven primarily by movement of the storeys further up the height of the wall.

Another reason necessitating optimization to the design process is the considerable increase in possible material and cross-sectional properties for a multi-storey shear wall design.

For ductility to be evaluated at the critical section, the software must wait for all other structural elements within the shear wall to calculate and return effective stiffness results to determine deflection at the top of the wall. This is a significantly longer process compared to the other design checks that have been implemented in previous versions of the software.

Figure 1 shows the overview of the MASS multi-storey design process. The main source of improvement comes from strategically saving key target variables from the first failed ductility verification which is completed at the end of the initial process and then checked in subsequent capacity design stages. This saves the time and computational effort of repeatedly performing all additional design steps in between.

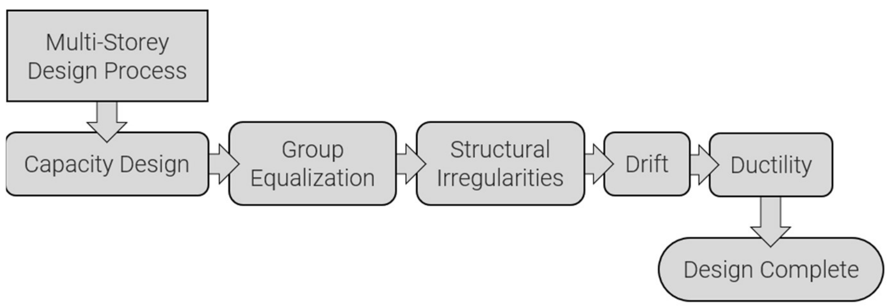


Figure 1: Overview of multi-storey shear wall design stages

MASS is likely considering only cross sections that are otherwise successful by this point since the ductility verification stage is done at the end of the design process, each attempt after a failure is nearly certain to take a long time because the software must repeat nearly the entire list of previously met requirements. This is a significant difference compared to design attempts that do not contain minimum reinforcement or have insufficient moment resistance as they are checked earlier on, and therefore losing much less progress that must be repeated each time the procedure is restarted. Before any optimization, designs were observed to take prohibitively long to execute and still often yielded unsuccessful design results in many cases.

ADDRESSING A FAILED DUCTILITY VERIFICATION

After a ductility verification has failed, two courses of action are possible depending on the shear wall cross section. A high-level summary can be seen in Figure 2.

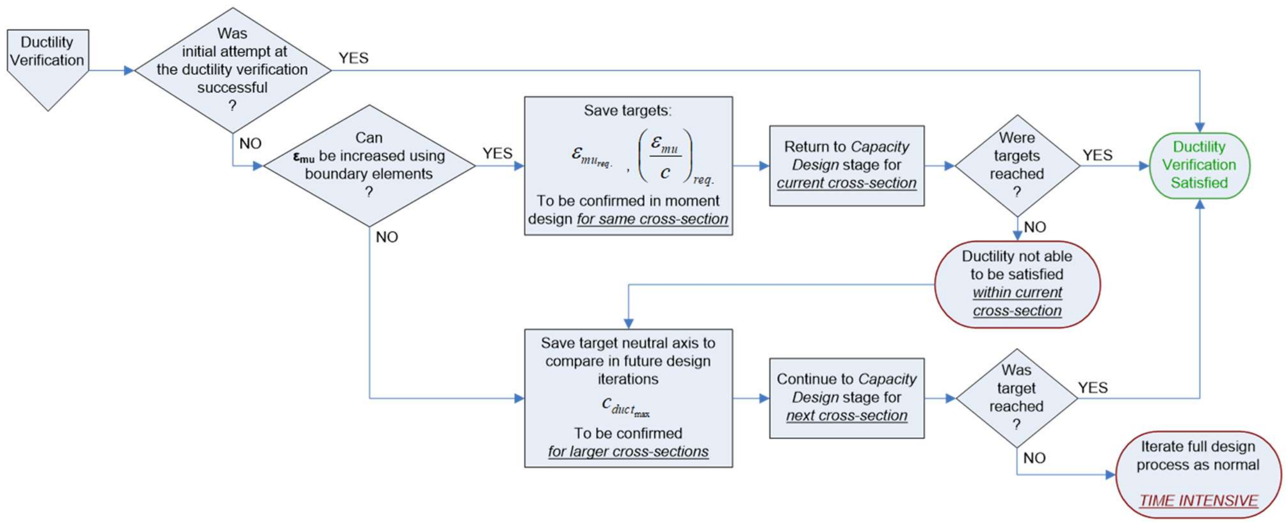


Figure 2: Simplified process diagram for addressing a failed ductility verification.

For the design of a typical shear wall with either a rectangular cross-section or one that contains flanges fails, there is nothing further that can be done without changing the cross-sectional properties of the wall. However, when a shear wall contains boundary elements and the vertical steel can be tied to resist compression, the same shear wall can be evaluated again taking a higher maximum compressive strain into account. The details on how the software manages both situations are described below.

SEARCHING FOR A NEW, ACCEPTABLE CROSS-SECTION

The most common course of action for contemporary design in Canada is to change the shear wall cross section as the existing rectangular layout is unable to facilitate higher maximum compressive strains, ε_{mu}.

In cases where the inelastic rotational demand, θ_{id}, is equal to the minimum specified in S304: 16.8.8.2[1] and ductility is still inadequate, the software determines the inelastic rotational capacity, θ_{ic}, and the associated neutral axis location that must be obtained. The variable $c_{duct,max}$ represents the largest possible distance between the compression face of the wall and the neutral axis that will yield a value of inelastic rotational capacity that meets demand, calculated as follows:

$$c_{duct_{max}} = \frac{\varepsilon_{mu} l_w}{2\theta_{id} + 0.004} \tag{1}$$

This value is saved in memory and checked during future cross-section *capacity design* stages where internal forces are balanced with externally applied axial loads to determine the updated neutral axis location. Before continuing through the capacity design stage and comparing the factored moment, M_f, to moment resistance, M_r, the software compares the neutral axis location, c, to $c_{duct,max}$ and triggers the failure message to alert the user of the following: "Design Fails: The compression zone under seismic loading is above the maximum that will result in a successful ductility verification in accordance with CSA S304-14: 16.8.8 and 16.9.7".

One thing to note is that this process is specific to designs where demand is equal to the minimum and assumes that demand will be only further reduced with cross-section property changes to address capacity. While not guaranteed, this is a reasonable safe assumption as stronger, stiffer cross-sections resulting from increased properties such as block size or strength or more frequently placed and larger reinforcing bars are likely to reduce drift and increase flexural overdesign, further reducing demand. Changes to the cross section causing larger compression zones and reduced capacity will not be considered as their neutral axis depth will also exceed $c_{duct,max}$. In the rare instance where demand equaling the minimum is no longer the case, c may be found to be below $c_{duct,max}$ which will satisfy the check at the moment design stage but when the ductility verification is done later on expecting to pass, demand is recalculated using updated drift and overdesign, at which point accurate values will still be compared at the expense of design time. Based on testing and experience, this assumption generally can be considered to hold true although there are exceptions.

Using the minimum demand values from S304: 16.8.8.2 [1], this can effectively be interpreted as permitting only the c/l ratios below 25% for moderately ductile shear walls and below 20.83% for ductile shear walls.

This is done for each load combination and the failure given for seismic load cases. Since this is completed at the very start of design for a section that, if found to be otherwise acceptable, will still not satisfy the ductility verification, significant time has been saved by identifying the failure early. This saves time by eliminating the need to equalize the design of shear wall element groups, receive effective stiffness data from other elements, and recalculate drift only to arrive at the same possibly anticipated conclusion, that the section exhibits inadequate ductility performance.

During the software development process, there were cases where implementing this proactive failure check reduced design times from nearly an hour to under a minute. Since lengthier design cycle times occur when the program struggles to find a passing solution, the added time is beneficial to the engineer as they can be alerted earlier on that other action is required and for other options to be considered.

INCREASING STRAIN USING BOUNDARY ELEMENTS

In addition to being able to reduce the compression zone through the use of reinforcement tied to resist compression, a benefit to designing shear walls containing boundary elements is the ability to consider and design for higher maximum compressive strains. While there are other benefits and trade-offs to consider, it is a considerable advantage to be able to increase θ_{ic} through increasing ε_{mu} in the event of a failed ductility verification. This can open up the option of satisfying ductility without having to otherwise increase cross section properties such as unit size or strength. The following process is used by MASS to address a failed ductility verification before the software moves on from the current otherwise successful cross-sectional design.

After a design iteration fails, the first thing the software does when boundary elements are present is solve for the maximum strain increase that would have satisfied the ductility verification.

Why not start at the maximum allowable ε_{mu}?
While maximizing capacity might be a reasonable place to start in satisfying the verification, there are trade-offs to higher expected compressive strain. Since increased damage is expected resulting from higher strains, S304: 16.10.4.1 [1] takes this into account by reducing the portion of factored shear resistance that can be contributed by the masonry alone as depicted in Figure 3.

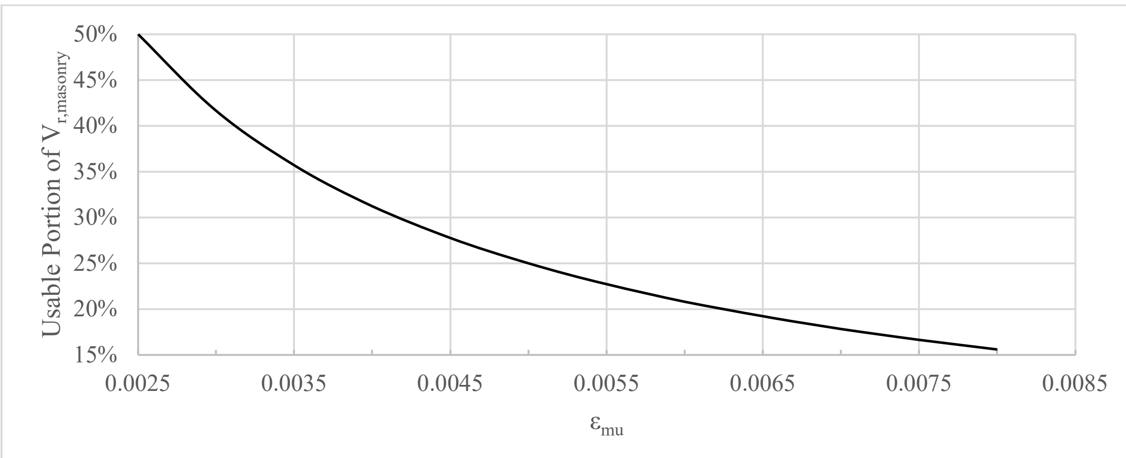


Figure 3: Increasing ε_{mu} reduces usable portion of $V_{r,masonry}$.

The reduction starts by using only 50% of the initial calculated masonry shear strength with increasing resistance lost for any additional increase. This is the main reason why ductility verifications are always attempted before assuming an increased strain – to make sure the 50% reduction is avoided if possible – as well as to design the wall with as little expected damage as possible for a satisfactory design.

How high is too high?
The degree to which ε_{mu} can be increased is limited by several code requirements referred to as "factors" within the design process.

1. ***Compressive strain within the web***

The first factor considered is the resultant strain in the shear wall web resulting from an increase in strain at the extreme compression face of the boundary element. This is a code requirement based on S304 16.10.2 and 16.11.2. Figure 4 shows the resultant compressive strain within the web resulting from an increase in strain at the extreme compression edge.

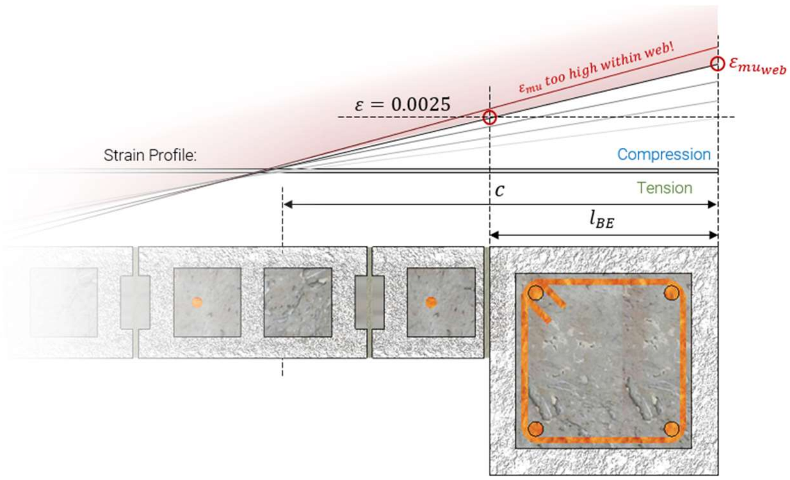


Figure 4: $\varepsilon_{mu,web}$ relationship with strain profile and cross-section geometry

While the boundary element is capable of resisting the increased compressive strain, the web of the wall remains limited to its initial value of 0.0025. The maximum compressive strain increase leading to the largest possible strain within the web, not to be confused with the largest strain occurring within the web, $\varepsilon_{req,web}$, is calculated as follows:

$$\varepsilon_{req_{web}} = \frac{0.0025}{\left(1 - \frac{l_{BE1}}{c}\right)} \tag{2}$$

Note that there are other code requirements regarding the minimum length of boundary elements that may restrict boundary element length. The target value for $\varepsilon_{req,web}$ only takes the current boundary element length into consideration.

2. ***Confinement of ties around compression reinforcement***

The second factor considered by MASS is the specified tie placement used to confine vertical reinforcing bars placed in the boundary element. S304: 16.11.3 provides two expressions that must each be evaluated where the larger area applies. Since only one of these expressions is a function of ε_{mu}, the software compares the minimum tie area to that expression and if they are not equal, the software concludes that any compressive strain increase is not a function of tie placement. If these two areas are found to be equal then the following rearranged expression is used to determine and save the largest maximum compressive strain increase that is a function of tie placement, $\varepsilon_{req,ties}$:

$$\varepsilon_{req_{ties}} = \frac{(n_l - 2) A_{sh} A_{ch} f_{yh}}{6 n_l A_g f'_m \, sh_c} - \frac{1}{300} \quad \text{if} \quad A_{sh_{\min}} = 0.2 k_n k_{p1} \frac{A_g}{A_{ch}} \frac{f'_m}{f_{yh}} sh_c \quad \text{where} \quad k_{p1} = 0.1 + 30\varepsilon_{mu} \tag{3}$$

If the largest possible ε_{mu} increase can be impacted by tie placement, it is saved as a factor for later comparison.

3. Absolute maximum strain

The final factor considered it based on S304: 16.10.2 which lists an absolute maximum value of ε_{mu} that can be considered. Any compressive strains larger than 0.008 are not permitted and saved as follows:

$$\varepsilon_{req_{\max}} = 0.008 \tag{4}$$

The software then compares the three factors and saves the smallest of these as the "governing" case, recorded as $\varepsilon_{req,gov}$. This represents the largest possible strain increase that will not violate any other shear wall requirements.

Determining the minimum strain increase required

After determining the maximum limit for a potential ε_{mu} increase, the next step is to solve for the strain that would have resulted in an inelastic rotational capacity that would have met or exceeded demand. This defined within the software as $\varepsilon_{req,duct}$ and is calculated using the following expression:

$$\varepsilon_{req_{duct.}} = \frac{c\left(2\theta_{1d} + 0.004\right)}{l_w} \tag{5}$$

If $\varepsilon_{req,duct}$ is found to be greater than the largest permissible governing increase, $\varepsilon_{req,gov}$, a successful ductility verification is not possible and MASS declares that design for that cross-section has failed. The software continues with the design process if other potential cross section candidates remain.

In the case where $\varepsilon_{req,duct}$ is less than the least of the factors considered in determining $\varepsilon_{req,gov}$, it is still possible to find a design that satisfies ductility. MASS then saves the required strain as well as the strain-to-c ratio. MASS then returns to the beginning of the design process with knowledge from previous failed ductility results and immediately checks what the new strain profile will look like and whether the section passes.

The problem that arises when ε_{mu} is increased is that the new strain profile will have an impact on the location of the neutral axis which was one of the inputs used to determine the required strain increase, $\varepsilon_{req,duct}$. In many cases, using the old c to determine the required strain would still result in a ductility failure due to the slight change in c after rebalancing all of the internal stresses to solve for c. Figure 5 shows various interaction diagram contours for a shear wall, demonstrating that for a given axial load, increasing strain moves the point on the failure envelope to a new location with a reduced compression zone.

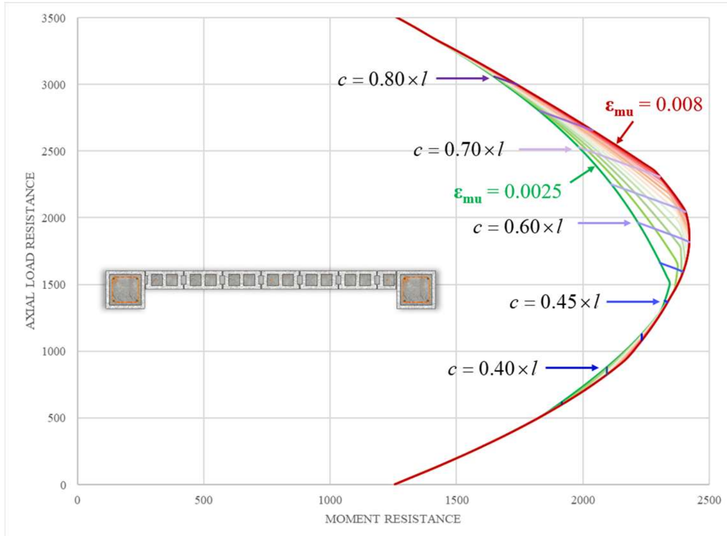
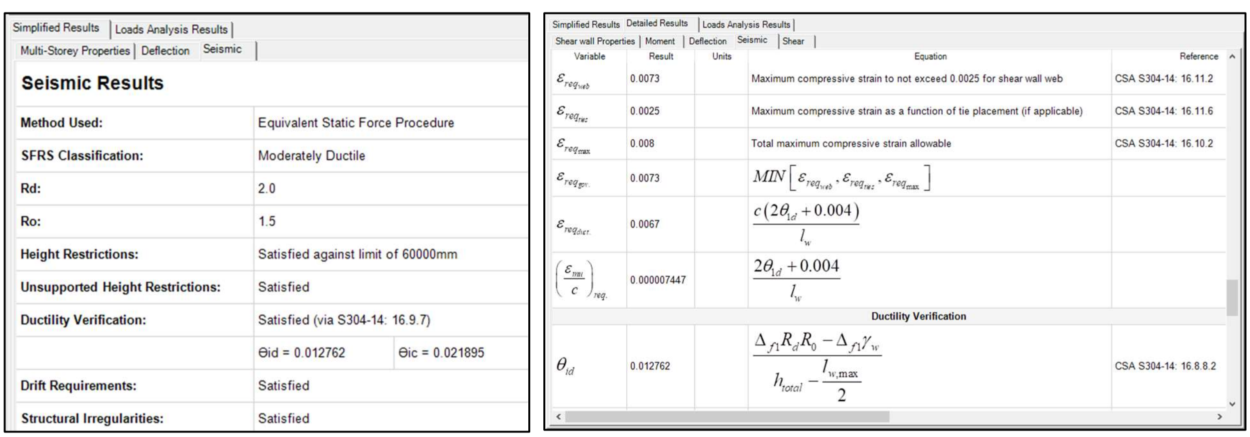


Figure 5: Increasing strain and reduction of c

The solution requires saving the strain to c ratio as its own target to later verify that it is also satisfied before leaving the capacity design stage. If the strain is adequate but the ratio is not then the software increments the strain slightly before re-checking the resulting ratio from the updated c until it is successful. If this can be done before reaching one of the factors previously discussed, it can be concluded that it is not possible to satisfy the ductility verification.

If the increased strain and resulting updated neutral axis location ratio satisfy the saved target value, the software will continue to perform all other scheduled design checks before reaching the ductility verification once again. By this point, it is guaranteed that the verification will be successful, but the process is still repeated for the purpose of populating the design calculation for the user to view in the detailed output, shown in Figure 6.



a) Simplified multi-storey results. b) detailed results in plastic hinge element

Figure 6: Ductility verification results shown in MASS

A benefit of this process of setting targets and comparing results is that this result can be obtained without spending extensive time iterating through calculations.

MAKING USE OF BOTH METHODS CONCURRENTLY

Shear wall designs making use of boundary elements can employ both design processes in the quest to satisfy ductility requirements. Within a given cross section, ε_{mu} is increased and if an increased strain is not successful, the cross-section properties can be iterated as would be done if there were no boundary elements, the strain increase process can also be attempted for each subsequent shear wall cross-section.

One difference when both design processes are combined is that neutral axis location targets described earlier, saved as $c_{duct,max}$, are not set since these are also a function of ε_{mu} which is variable for boundary elements with tied compression steel. If strain factors can be used to set targets, that method is used. When that option is not available, neutral axis targets are used.

RECOMMENDATIONS FOR ADDRESSING INADEQUATE DUCTILITY

While these processes have dramatically reduced the amount of time needed to either find a successful design or declare failure, there is more that can be done.

Consider ALL masonry unit options

Within MASS, while it may speed up the design process to reduce the number of selections when the designer has a pretty good idea of the unit size or strength they expect to use, it may be worth opening up the selections to increased block size or strength, allowing the wall to resist the same internal compression force within a smaller region of the wall. Another option to consider is prism testing so that a higher assemblage material strength can be used in design calculations.

Model flanges or boundary elements where possible

The benefit of flanges in the case of satisfying ductility is that having a compression flange dramatically reduces the length of wall required to resist the applied loading. While not always possible, many designers prefer to separate walls into rectangular segments and avoid the hassle of detailing connections and checking shear flow capacity across the flanged interface. If encountering difficulties with ductility, the engineer can consider modelling the walls as being flanged and connected, increasing flexural resistance and more importantly, improving the inelastic rotational capacity.

Boundary elements offer all the same benefits seen in flanges with two notable improvements beyond the increases to strength and stiffness. While the compression zone is concentrated within a smaller portion of masonry due to the change in cross section thickness near each end of the wall, the compression force resisted by tied vertical reinforcement further reduces the compression zone by a significant amount. Additionally, the ability to increase ε_{mu} is also helpful in increasing θ_{ic} to meet θ_{id}.

Address factors behind the governing strain increase (boundary element designs only)

If inelastic rotational capacity is limited by potential ε_{mu} increases that are governed by strain within the shear wall web, the designer may find it useful to re-examine the cross-section geometry. Increasing boundary element length moved the largest compressive strain within the web further along the strain profile and is effective in opening up higher strains to reach a greater inelastic rotational capacity.

This is something that the software will not do on the user's behalf which underlines the importance of the user's professional engineering judgement within the use of tools to assist with masonry design.

Reconsider the building layout

If there is flexibility in where loadbearing walls are placed, the layout can be improved to spread the axial load over a higher number of walls. Any structure with vertically continuous partition walls such as between dwelling units can be designed as loadbearing shear walls, reducing the compression area in each wall. Consider designing flooring systems to bear upon walls that are not adjacent to as many openings or movement joints, reducing the compression zone for the smaller shear walls within the SFRS.

Consider alternatives after the first successful result

The MASS design algorithm tries to balance economics by incrementing reinforcement before block size but there are always drawbacks. For example, designs using smaller units and a large area of reinforcement will always be shown when possible before options with a larger or stronger unit with significantly less steel. It is important to consider this and play around with the selections after seeing the first successful result because it can be a function of the algorithm which does not always result in an optimal design.

The role of professional engineering judgement cannot be understated when using a software tool within the structural design process. The ductility verification is one area in particular which illustrates its importance.

ACKNOWLEDGEMENTS

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REFERENCES

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