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EVALUATING SEISMIC PERFORMANCE OF SHEAR WALL BUILDINGS USING CSA A23.3-2014: A FRAMEWORK

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ABSTRACT: The greater Vancouver area has a large number of high-rise concrete shear wall buildings that were constructed before the advent of modern seismic design requirements. Many of these buildings have thin concrete shear walls without minimum ties at the ends. The methods currently available for evaluating the seismic performance of buildings are not well suited for these buildings. The 2014 edition of CSA 23.3 contains state-of-the-art rational analysis procedures for shear wall buildings that can be used to evaluate seismic performance of existing buildings. Accurate estimates of shear wall displacements can be made using three-dimensional linear dynamic analysis as long as an appropriate effective stiffnesses is used for the concrete shear walls that reflects the level of expected damage. A rational procedure is provided for estimating strain demands in shear walls due to the displacement demands. Refinements are possible for the elastic component of displacements and curvatures, the plastic hinge length and the compression strain limits. CSA A23.3-2014 contains new procedures for estimating the seismic demands on gravity-load components of the building. These procedures account for the increased demands from foundation rotations, increased interstory drifts due to nonlinear shear deformation of walls in the flexural hinge region, and the nonlinear distribution of shear wall flexural deformations over the height of the building.

1. Introduction

New concrete shear wall buildings are designed according to the National Building Code of Canada (NBCC) and the requirements of Canadian Standard CSA A23.3. The process involves: (i) selecting the ductility level of the shear walls – conventional, moderately ductile or ductile; (ii) calculating the force demands according to NBCC using the corresponding force reduction factors R_d R_o that are prescribed for the system, and; (iii) finally designing a concrete shear wall that meets all the design requirements in CSA A23.3 for the selected system type. The design requirements include various capacity design requirements and detailing rules that vary depending on the selected ductility level of the system.

When designers are faced with the task of evaluating an existing building using NBCC and CSA A23.3, the approach they normally take is to begin by trying to decide what ductility level (conventional, moderately ductile, or ductile) is appropriate for their building based on the level of existing detailing in the shear walls. Invariably the detailing does not meet modern design requirements for ductile shear walls and the designer will often characterize the building as conventional construction (R_d R_o = 1.5 x 1.3 = 1.95). The requirements for conventional construction shear walls are continually being increased and most existing shear walls do not meet the requirements for conventional shear walls according to the 2014 edition of CSA A23.3. One solution is for designers to characterize their shear walls as an 'Other Concrete SFRS' (R_d R_o = 1.0 x 1.0 = 1.0), which would mean a doubling of the design forces.

Once the shear walls have been characterized and the force reduction factors are defined, the seismic evaluation often involves calculating the factored forces according to NBCC, calculating the factored resistances according to CSA A23.3, and finally calculating how much the factored resistances are less than the factored forces. If the factored resistances are below a certain percentage of the factored forces, the structure is retrofit to increase the seismic resistance of the building to within an acceptable percentage of the factored forces.

This paper suggests a much more rational procedure to evaluate the seismic performance of concrete shear wall buildings using NBCC and the 2014 edition of CSA 23.3. The latter document contains many state-of-the-art rational analysis procedures in addition to prescriptive design rules. These rational analysis procedures can be used to make a displacement-based assessment of the seismic performance of shear wall buildings.

2. Overview of Proposed Procedure

Figure 1 gives a "road map" of the evaluation procedure. The first step is to define the hazard level. New design is normally done for the mean 2% in 50-year hazard level as defined in NBCC. It may be appropriate to conduct a seismic evaluation for a lower seismic demand as a higher probability of collapse may be acceptable. For many structures, the most important issue may be excessive damage when subjected to lower hazard levels. The general procedure described here can be used to assess the structure for collapse or for damage under a service-load level earthquake.

Figure 2 summarizes how the displacement demands vary for Vancouver (City Hall) at different hazard levels.

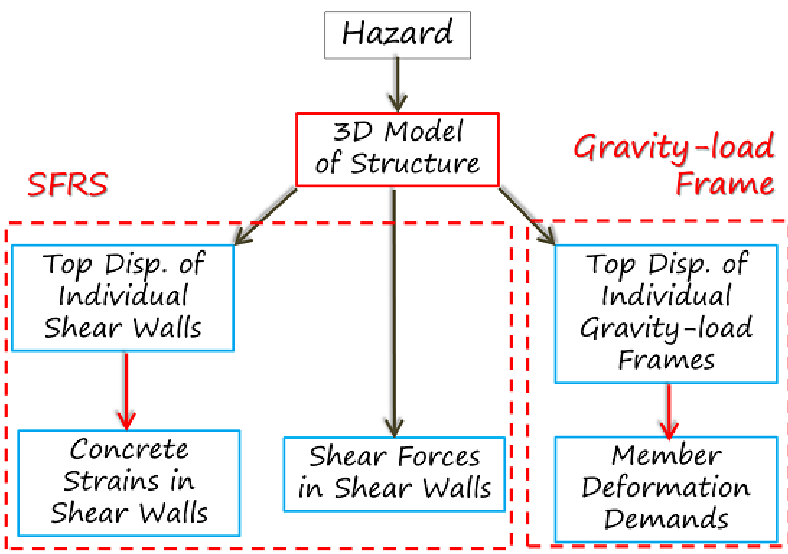

Fig. 1 – Overview of proposed evaluation procedure.

3. Displacement Demands

Given the defined hazard in terms of a spectrum, the first step is to determine the displacement demands in the structure using a three-dimensional linear dynamic analysis. A linear response spectrum analysis (RSA) can provide very accurate estimates of the displacement demands at the top of a concrete shear walls as long as the appropriate effective flexural rigidity EI_e is used. Adebar and Dezhdar (2015) conducted a detailed study to develop an expression for the average EI_e that should be used in RSA of concrete shear walls accounting for cracking. The expression has been adopted as Eq. 21.2 in Clause 21.2.5 of CSA A23.3-2014.

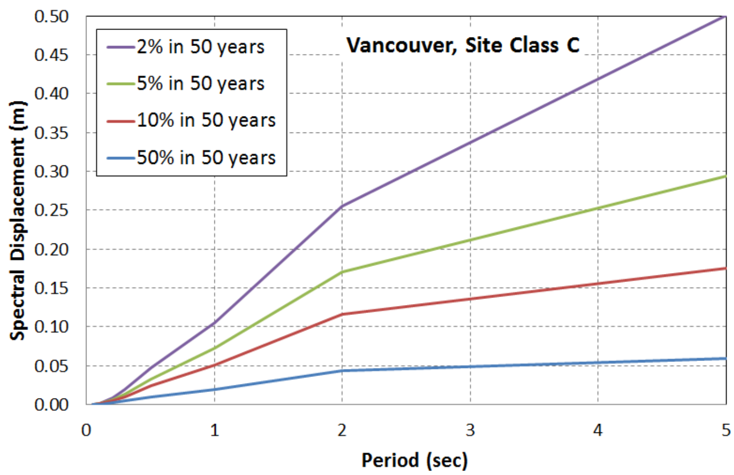

Fig. 2 – Relative displacement demands for different hazard levels.

The expression includes the term $R_d R_o / \gamma_w$; however $R_d R_o$ does not need to be defined to use the expression because the value of γ_w varies depending on what $R_d R_o$ is selected. γ_w is the wall overstrength factor equal to the ratio of the load corresponding to nominal moment resistance of the wall system to the factored load on the wall system (calculated using $R_d R_o$), but need not be taken as less than 1.3. The term $R_d R_o / \gamma_w$, which appears in many other places in CSA A23.3-2014, can be replaced with the simple ratio $R = M_e / M_n$, where M_e is the bending moment demand at the base of the shear wall (where yielding will start) calculated from the RSA using the appropriate value of EI_e, and M_n is the nominal moment capacity of the wall.

The value of the effective flexural rigidity EI_e reduces from 100% to 50% of the flexural rigidity of the uncracked section EI_g as the elastic bending moment demand increases relative to the nominal bending moment capacity. Thus an appropriate approach for evaluation is to assume the lower-bound value of EI_e = 0.5 EI_g, which occurs if the elastic bending moment M_e is greater than or equal to 2.4 times the nominal bending moment capacity (Adebar and Dezhdar, 2015). If the elastic bending moment demand is less than 2.4 times the nominal bending moment capacity, an increased value for EI_e can be used. Iteration may be needed to determine the largest value of EI_e which results in the minimum displacement demands.

The three-dimensional RSA is to be used to determine the displacements at the top of each shear wall and at the top of each gravity-load frame. This information is used to evaluate the shear walls and the gravity frames as shown in Fig. 1.

4. Evaluating Strain Demands on Shear Walls

An important part of the evaluation is ensuring the concrete shear walls can tolerate the strain demands. This includes a tension strain limit to ensure the reinforcement will not fracture in tension, and a compression strain limit to ensure the concrete will not crush in compression. CSA A23.3-2014 contains a rational analysis procedure for ensuring concrete shear walls have adequate strain capacity. The background to the procedure is given by Adebar, Mutrie and DeVall (2005).

Dezhdar (2012) did a detailed evaluation of the CSA A23.3 method for evaluating strains in concrete walls and found the method is safe. He also made a number of recommendations on how the method could be refined – made less conservative. Based on Adebar, Mutrie and DeVall (2005), CSA A23.3 uses an upper-bound estimate of the yield curvature equal to $0.004/l_w$. Dezhdar (2012) found that $0.003/l_w$ is an appropriate upper-bound estimate of the yield curvature and it is recommended that this refined value be used in evaluation of existing shear walls.

Based again on the recommendation of Adebar, Mutrie and DeVall (2005), the CSA A23.3 approach for estimating base curvature demands assumes that the elastic potion of total roof displacement demand is equal to the total displacement demand divided by bending moment ratio $R = M_e / M_n$ (see Section 3). Thus if the nominal bending moment capacity M_n is equal to the elastic bending moment demand M_e from RSA, the elastic portion of roof displacement is equal to the total displacement, whereas if the nominal bending moment capacity is half the elastic demand, half the total displacement is considered to be elastic. Dezhdar (2012) confirmed that this approach is generally safe. Some refinements are possible for reducing the inelastic portion of the total displacement demand.

The assumed plastic hinge length in the wall l_p is an important parameter for converting displacement demands to curvature demands. The plastic hinge length is the length (height) over which the inelastic curvatures in the wall are assumed to be uniform. When converting top-wall displacement to hinge rotation, CSA A23.3 assumes the plastic hinge length is equal to the wall length ($l_p = l_w$), while when converting inelastic hinge rotations to inelastic curvatures, the plastic hinge is assumed to be half the wall length ($l_p = 0.5l_w$). Bohl and Adebar (2011) conducted a detailed study using nonlinear finite element and recommended the following expression for the length over which the inelastic curvatures can be assumed to be uniform:

$$l_p = \left(0.2l_w + 0.05z\right)\left(1 - 1.5P / f_c'A_g\right) \leq 0.8l_w \tag{1}$$

where $z = M/V$ in the plastic hinge region of the wall; and a compressive axial load P is taken as positive.

Eq. (1) indicates that the plastic hinge length reduces as the axial compression applied to the wall is increased. Eq. (1) is meant for a wall where the vertical reinforcement yields in tension prior to the concrete crushing in compression, which will always be the case for new construction. If a concrete shear wall may fail by concrete crushing prior to vertical reinforcement yielding, a smaller plastic hinge length, perhaps as small as $0.10l_w$, must be used. Further work is needed to establish the appropriate plastic hinge length for walls failing in compression.

Finally, the last parameter that influences the displacement capacity of the concrete shear walls is the compression strain limit for concrete. CSA A23.3 assumes a compression strain limit of 0.0035 for assessing whether a concrete wall with at least column ties at the end of the wall requires further confinement reinforcement. When evaluating existing concrete shear walls, a range of compression strain limits need to be used. For the case of thin walls without minimum column ties at the end of the wall, a maximum compression strain of 0.002 should be used (Adebar, 2013), especially if it is a long wall that will not experience significant strain gradient. On the other hand, if a wall is relatively thick and contains a good arrangement of ties, and will be subjected to significant strain gradient, than a compression strain limit as high as 0.005 is appropriate when conducting an evaluation of an existing building.

Table 1 – Concrete compression strain limits.

0.002	0.0035	0.005
thin walls; no ties; low strain gradient	assumed in CSA A23.3	well-detailed sections; significant strain gradient

5. Evaluating Shear Force Demands

The linear dynamic (response spectrum) analysis provides estimates of the elastic shear force demands applied to the concrete shear walls. For design, these shear forces have typically been reduced by the same force reduction factor as is used for the bending moment based on the flexural ductility. It has been known for some time that this approach is unsafe; but a simple solution appropriate for use in Canada was not readily available.

CSA A23.3 has recently added a new clause (21.5.2.2.7) to account for the magnification of shear due to inelastic effects of higher modes. The background to this new clause is given by Adebar, Dezhdar and Yathon (2015). While designers that use nonlinear response history analysis (with a linear shear model) typically use a shear amplification factor of up to 3.0, the shear amplification factor in CSA A23.3-2014 is limited to 1.5. This was done because concrete shear walls have considerable shear ductility, the maximum shear force demand only occurs in one short cycle and the shear demand in all other cycles is considerably less, and the maximum shear force does not occur simultaneously with the maximum base rotation, while the calculation of shear resistance assumes that it does.

The shear amplification factor in CSA A23.3-2014 is a function of $R_d\ R_o\ /\ \gamma_{w,}$ which as described in Section 3, is analogous to the bending moment ratio $R = M_e\ /\ M_n$.

6. Evaluating Demands on Gravity-Load Frames

Observations from past earthquakes have shown that the collapse of buildings is often triggered by failure of structural members that are not part of the seismic-force-resisting system (SFRS), such as gravity-load columns. As a result, model building codes such as the National Building Code of Canada (NBCC) require that all structural members not designated as a part of the SFRS be designed to support the gravity loads while subjected to the design seismic displacements.

Figure 3 provides a simple schematic drawing of a typical building system. The SFRS (shear walls in this case) may only support a small portion of the weight of the building. By definition, the SFRS must be stiff enough so that the majority of the lateral seismic force is resisted by the SFRS; but since the SFRS and the gravity-load frame are tied together, both will experience the same lateral displacements. Any movement of the soil below the SFRS will help to protect SFRS; but will result in increased displacements of the SFRS, which will increase the demands on the gravity-load frame.

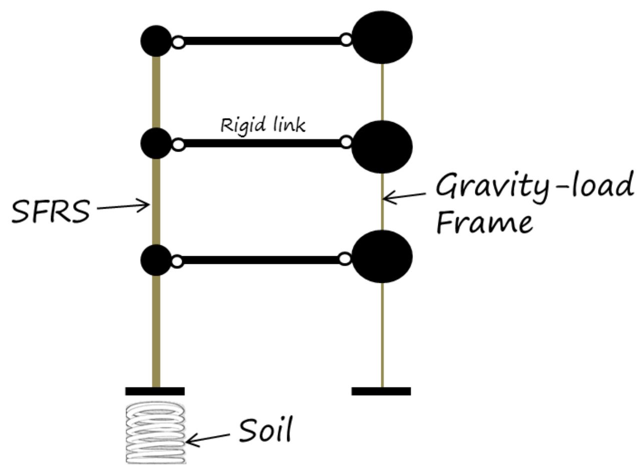

Fig. 3 – Simple schematic drawing of a building system: if gravity-load frame is not sufficiently flexible or ductile, collapse of the structure may result; soil movement isolates the SFRS but increases demands on the gravity-load frame.

An analysis must be done to determine the forces and deformations induced in the gravity-load resisting frame members due to the seismic deformation demands on the SFRS. In concept, this involves displacing the complete structure – SFRS and gravity-load resisting frame – to the design displacement.

The yielding that occurs in the SFRS before it reaches the design displacement causes a concentration of deformation demands at plastic hinge locations. The influence of the resulting inelastic displacement profile of the SFRS must be accounted for when determining demands on the gravity-load frame. The best way to accomplish this is to do a nonlinear analysis of the complete structure; however, a linear model with appropriately reduced section properties at plastic hinge locations can be used to estimate the inelastic displacement profile for shear wall buildings (Dezhdar, 2012).

An important issue is what stiffness should be used for the gravity-load resisting frame members. Lower-bound estimates of effective stiffness are commonly used for the SFRS to make a safe estimate of the design displacement; but higher estimates of effective stiffness of the gravity-load resisting frame members must be used to make a safe estimate of the forces induced in these members by the inelastic displacement profile of the SFRS.

Most Canadian designers do not use nonlinear analysis for the design of concrete buildings. Thus a simplified analysis procedure was developed for gravity-load resisting frames in shear wall buildings. The simplified analysis procedure was developed from the results of numerous nonlinear analyses of shear wall buildings (Dezhdar and Adebar, 2015). It relates the design displacements of the SFRS at the top of the building to the profile of deformation demands over the height of the building.

Figure 4 shows various envelopes of interstory drift ratios as a ratio of the global drift ratio Δ/h_w. Δ is the design lateral deflection at the top of the gravity-load resisting frame, in the principal direction of the frame, and h_w is the height of the building measured to the base of the plastic hinge zone in the SFRS.

The envelope that has been adopted in Clause 21.11 of CSA A23.3-2014 is shown as a solid red line. This envelope is suitable for a building with a shear wall that has considerable inelastic action in the plastic hinge region.

The red dashed lines show a refined envelope that could be used. The interstory drifts due to the flexural deformation of the wall reduce to zero at the base of the wall, assuming the base of the wall to be perfectly fixed. The vertical red dashed line defines the interstory drifts resulting from shear strains in the wall. This is discussed further below.

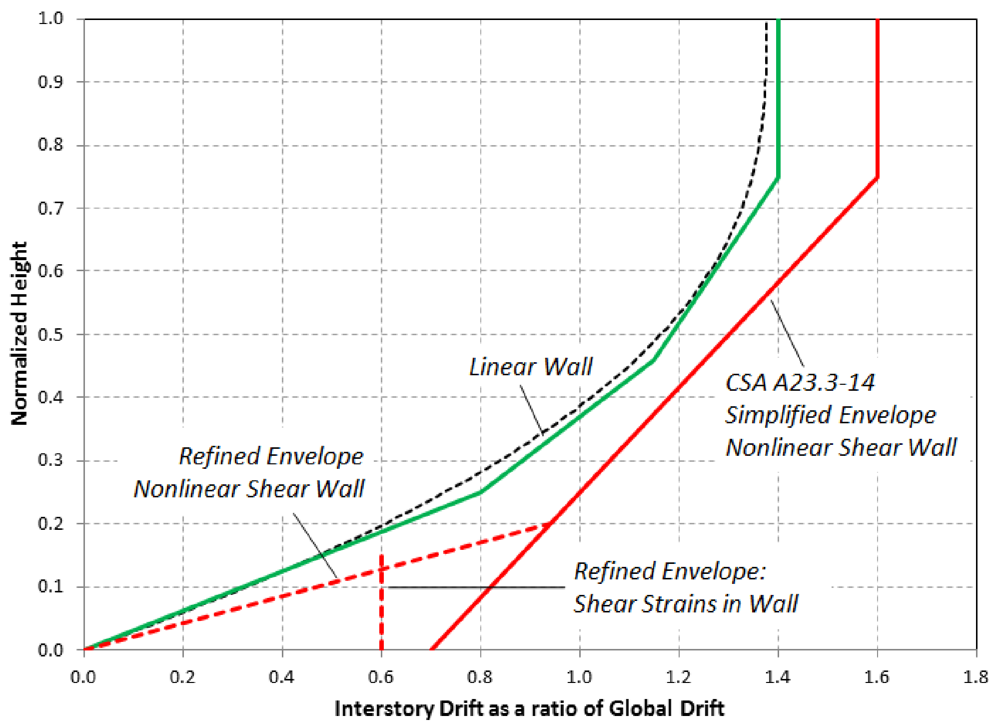

Fig. 4 – Interstory drift envelopes defined in terms of the global drift ratio Δ/h_w.

The black dashed line shows the results from a linear dynamic analysis. The maximum interstory drift at the top of the wall is 1.4 times the global drift. The green line indicates a piece-wise linear envelope of interstory drifts that can be used when a cantilever wall remains linear – i.e., no yielding at the base. As the yielding at the base of the wall increases, the envelope transitions to the red line given in CSA A23.3-2014. For evaluation purposes, it makes sense to interpolate between these envelopes depending on the level of inelastic action at the base of the wall.

A gravity-load resisting frame may consist of a single column or bearing wall interconnected to a shear wall by slabs (and beams); or may consist of many interconnected columns and/or walls. There are usually several different gravity-load resisting frames in each direction of a building.

There are a number of different ways that the envelope in Fig. 4 can be used to determine the demands on the gravity-load resisting frame. When the gravity frame has unique features like a large transfer girder at one or perhaps a few levels, a computer model of a small portion of the frame can be developed and then subjected to displacements that result in the interstory drift given in Fig. 4 for the height of the floor level. It is also possible to develop a computer model of the entire gravity frame and subject it to a single displacement profile that results in the envelope of interstory drifts at all levels. Further information is given by Adebar DeVall and Mutrie (2014).

6.1. Wall Shear Strains

Shear strains generally have negligible influence on maximum displacements at the top of slender shear walls; but may significantly increase interstory drift ratios at lower levels where gravity-load columns are often less flexible. A nonlinear FE model calibrated with experimental results confirmed that large shear strains occur in flexural tension regions of concrete walls due to vertical tension strains in the presence of diagonal cracks and in the absence of demand on the horizontal shear reinforcement (Bazargani and Adebar, 2015). A “fan” of diagonal cracks will form at the base of flexurally hinging walls independent of the shear stress level.

A parametric study revealed that a principal strain angle of 75 deg. can be used to estimate shear strains from vertical tension strains. Thus interstory drift ratio due to shear strains in the wall, which is equal to the shear strain, can be estimated very simply as 60% of the average vertical strain at the mid-length of the wall as follows:

$$\gamma = 0.6\,\bar{\phi}\,(0.5 l_w - c) \qquad (2)$$

where: $\bar{\phi}$ is the average curvature at the centre of the story height; l_w is the length of the wall and c is the distance from the compression face of the wall to the neutral axis. The methods discussed earlier can be used to estimate the curvature of the wall.

Alternatively, a simple and safe estimate of interstory drift ratio due to shear strains can be made by assuming the average vertical strain in the wall is equal to the global drift. This result follows from assuming the flexural displacement at the top of a wall is entirely due to uniform curvature over a plastic hinge length equal to half the wall length and by assuming the compression strain depth c is negligible. In that case, the shear strain, which is equal to the interstory drift, is equal to 60% of the global drift ratio. This corresponds to the dashed red vertical line shown in Fig. 4.

6.2. Foundation Rotations

When a foundation is capacity protected by the SFRS, the rotation of the foundation may be determined using a static analysis that accounts for the assumed bearing stress distribution in the soil or rock and the stiffness of the soil or rock. Adebar (2015) developed such a method that requires two parameters to be determined from design charts. The first parameter reflects the initial linear rotational stiffness of foundations based and depends only on the footing geometry. The second parameter reflects the additional rotations due to displacements at the “toe” of the footing from the reduction in shear modulus of the soil. It depends on the magnitude of the bearing stress as a ratio of the factored bearing strength, and the length-to-width ratio of the bearing stress distribution.

A simplified version of the method was presented where the two parameters are estimated from equations, and a third, further simplified, version was presented in the form of a single equation. Eq. (3) has been adopted in CSA A23.3-2014, and the other versions will be presented in the commentary as more accurate methods for estimating foundation rotation. The more simplified methods give larger rotations than the simplified methods – the less simplified methods can be used to reduce the estimate of rotation.

When the applied overturning moment is sufficiently large to cause the “heel” of a footing to up-lift, $M_f \geq P_f\, l_f/6$, the rotation of the foundation can be estimated from:

$$\theta = 0.3\left(\frac{q_s}{G_0}\right)\left(\frac{l_f}{a_s}\right)\left\{1 + 2\left(\frac{a_s}{b_f}\right)^{1.5}\right\} \qquad (3)$$

where a_s is the length of uniform bearing stress in soil or rock required to resist the factored overturning moment M_f applied to the foundation, given the footing length l_f (perpendicular to axis of rotation). q_s is the magnitude of uniform bearing stress in soil or rock required to balance the applied vertical load P_f, given the footing width b_f (parallel to axis of rotation). G_o is the initial Shear Modulus of soil or rock, which may be estimated in kPa units as $\gamma_s V_s^2/1000$ when γ_s, the density of soil or rock, is in kg/m3, and V_s, the shear wave velocity measured in the soil or rock immediately below the foundation, is in units of m/s. These terms are shown in Fig. 5.

When the applied overturning moment is smaller than what causes the "heel" of the footing to uplift, the approach to be used to estimate the foundation rotation at $M_f = P_f\ l_f/6$, and then assume the rotation at smaller overturning moments are proportionally less.

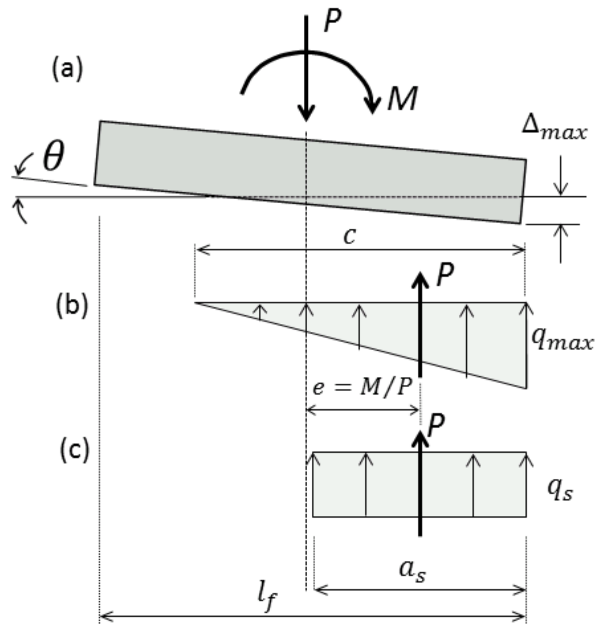

Fig. 5 – Movement of foundations: (a) rotation of footing subjected to vertical load and overturning moment; (b) actual bearing stress distribution in the linear soil range; (c) statically equivalent uniform bearing stress.

Foundation flexibility will generally reduce the seismic loads that must be resisted by a structure; but will increase the displacements of the structure. A simplified approach that can be used, and has been adopted into CSA A23.3-2014, is to assume that the interstorey drift ratios of the building determined from a fixed-base model is increased at every level, including immediately above the footing, by an interstorey drift ratio equal to the footing rotation.

7. Summary

The 2014 edition of CSA A23.3 contains many rational analysis procedures for concrete shear walls that can be used to perform a seismic evaluation of existing shear wall buildings.

It is not necessary to make any sort of assumption about the ductility of the shear walls when starting the evaluation procedure because the provisions in CSA A23.3-2014 include the term $R_d R_o / \gamma_w$, and γ_w is a function of $R_d R_o$. When doing an evaluation it is convenient to replace the term with the simple ratio $R = M_e / M_n$, where M_e is the bending moment demand at the base of the shear wall calculated from a linear dynamic analysis and M_n is the nominal bending moment capacity of the wall.

The main components of the seismic evaluation include: (i) evaluating the displacement capacity of the shear walls, (ii) evaluation the shear force demands on the shear walls, (iii) evaluation the displacement demands on the gravity-load frame.

The displacement capacity of the shear walls is determined by comparing strain demands at the base of the wall with the compression strain capacity of concrete. The refinements that should be made when doing an evaluation using CSA A23.3-2014 are: reduced yield curvature, reduced inelastic portion of total displacement, increased plastic hinge length, modified concrete compression strain capacity depending on the detailing conditions at the end of the wall.

CSA A23.3-2014 includes new procedures for evaluating the deformation demands on the gravity-load frame. A refined interstory drift envelope was presented to allow refined calculations to be done for evaluation of existing buildings. The demands on the gravity-load frame must consider the influence of shear strains over the plastic hinge region and the influence of foundation movements reducing the demands on the shear walls; but increasing the overall demands on the gravity-load frame.

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