DESIGN OF HIGH-RISE CORE-WALL BUILDINGS: A CANADIAN PERSPECTIVE

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ABSTRACT :

Presents the background to prescriptive design procedures that have recently been developed to permit the safe design of high-rise core-wall buildings using only the results of response spectrum analysis (RSA) at the MCE level of ground motion. This includes: (i) effective stiffness of walls to estimate displacement demands at top of wall using RSA, (ii) procedures to estimate rotation demands on cantilever walls and coupled walls, (iii) procedures to estimate rotational demands on coupling beams, (iv) procedures to ensure the flexural hinge regions of the wall have adequate shear capacity, (v) procedure to design for shear reversal below flexural hinge due to stiff floor diaphragms connected to perimeter foundation walls, and finally (vi) some discussion on dynamic magnification of shear.

KEYWORDS: Building codes, high-rise buildings, reinforced concrete walls, seismic design.

1. INTRODUCTION

Concrete core-walls (see Fig. 1) have been the most popular seismic force resisting system in western Canada for many decades, and recently have become popular on the west coast of the US for high-rise buildings up to 600 ft (180 m) high. Without the moment frames that have traditionally been used in high-rise concrete construction in the US, the system offers the advantages of lower cost and more flexible architecture. In the US, such buildings are currently being designed using nonlinear response history analysis (NLRHA) at the Maximum Considered Earthquake (MCE) level of ground motion. In Canada, these buildings are designed using only linear dynamic (response spectrum) analysis at the MCE hazard level combined with various prescriptive design procedures. This paper presents the background to some of the prescriptive design procedures that have recently been developed to permit the safe design of high-rise core-wall buildings using only the results of response spectrum analysis (RSA).

The design requirements for earthquake loads and effects are given in the National Building Code of Canada (NBCC). The 2005 edition of NBCC was extensively revised from the previous edition. The design-basis earthquake is the maximum considered earthquake with a 2% in 50-year probability of exceedance (2500 year return period). Table 1 summarizes the concrete wall systems defined in the 2005 NBCC and the corresponding seismic force reduction factors: a ductility force modification factor R_d, and an overstrength force modification factor R_o. The specific design and detailing requirements for concrete systems are given in Clause 21 of the Canadian concrete code (CSA Standard A23.3 – Design of Concrete Structures). A number of the provisions in that code for the seismic design of concrete wall buildings were completely revised for the 2004 edition. Many of these provisions are displacement-based, and thus require an estimate of the lateral displacement of the building due to the design-basis earthquake.

Table 1: Concrete wall seismic force resisting systems (SFRS) in Canada.

Type of SFRS	R_d	R_o	$R_d \times R_o$	Limitation
Ductile Coupled Walls	4.0	1.7	6.8	-
Ductile Partially Coupled Walls	3.5	1.7	6.0	-
Ductile Shear Wall	3.5	1.6	5.6	-
Moderately Ductile Shear Wall	2.0	1.4	2.8	60 m height*
Conventional Shear Wall	1.5	1.3	2.0	30 m height*

* Except in low seismic regions

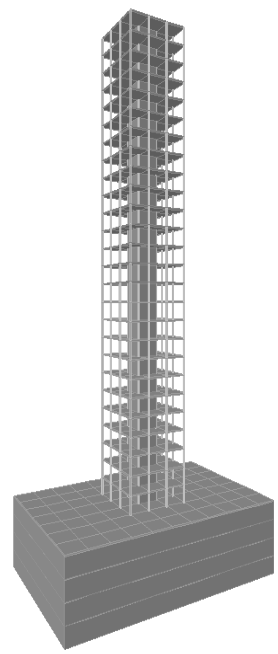

Fig. 1 Core-wall building.

2. EFFECTIVE STIFFNESS OF WALLS

The lateral displacement of concrete walls due to the design-basis earthquake must be determined using a structural model that accounts for the effect of concrete cracking. The effective stiffness used in linear seismic analysis is normally determined from the uncracked section stiffness multiplied by a single factor that accounts for variation of cracking over the height, and variation of cracking with increasing lateral load prior to reinforcement yielding. For example, the flexural rigidity $E_c I_e$ is normally taken as $\alpha E_c I_g$, where one value of α is used for an entire structure. FEMA 356 recommends α = 80% for previously uncracked concrete walls and α = 50% for previously cracked walls. The commentary to the 1995 New Zealand concrete code recommends α = 25% for a concrete wall with no axial force and α = 35% for a concrete wall with an axial compression force equal to 10% of $f_c' A_g$.

The most general model for calculating nonlinear flexural response of any reinforced concrete section is a fiber (layer) model, in which stress-strain relationships for concrete and reinforcement are used to determine the stress in fibers (layers) of the cross-section at a given strain level. The results from a fiber model were used to develop a simple nonlinear flexural model for typical high-rise concrete shear walls where the nonlinear compression strains of concrete are relatively small (Adebar and Ibrahim, 2002). The simple model is well suited for nonlinear static (pushover) analysis of concrete walls, and leads directly to the development of a linear (effective stiffness) model for high-rise concrete walls. A large-scale test of a slender core-wall was done to calibrate the nonlinear model (Adebar, Ibrahim and Bryson, 2007).

The response of a concrete wall during any load cycle depends on the maximum deformation amplitude in previous cycles. Ibrahim and Adebar (2004) presented simplified expressions for the upper-bound and lower-bound response corresponding to previously uncracked and severely cracked walls. The expressions are a function of only the axial compression force at the base of the wall. For any particular wall and design basis earthquake, the effective stiffness was expected to be between the two bounds. In the initial draft of the 2004 Canadian concrete code, the upper-bound reduction factor was proposed for moderately ductile (R_d = 2.0) walls and the lower-bound reduction was proposed for ductile (R_d = 3.5) walls. The rationale was that walls designed with a larger force reduction factor would be more likely to be severely cracked during an earthquake. The 2005 NBCC overstrength force reduction factors (shown in Table 1) maintain the design force levels similar to historical levels even though the design earthquake has been significantly increased; however these factors have no compensating effect on the design displacements. Due to concerns that the combined effect of larger design-basis earthquakes and drastically reduced effective stiffnesses would have too much of an effect on final designs, it was decided to use the upper-bound effective stiffness for all concrete walls until such time as better information is available on the choice of effective stiffness between the upper and lower bounds.

Nonlinear response history analysis was used to investigate the effective stiffness of high-rise cantilever walls. A simplified force-displacement model for high-rise cantilever walls was developed from the results of a fiber model, the simplified trilinear bending moment-curvature model, and results from the large-scale test of a cantilever wall. The model consists of a rational force-displacement envelope, and empirical hysteretic rules. The force-displacement envelope of a concrete wall depends on wall geometry, axial compression force, and amount of vertical reinforcement. The practical range of these parameters was used to establish the practical range in force-displacement envelopes normalized by wall strength (expressed as ratio of elastic demand to strength of wall *R)*, and normalized by initial stiffness of wall. The lower-bound force-displacement relationship was assumed to be trilinear. The initial linear portion extended 20, 40, 50, 60 or 80% of the wall strength, and these were called L2, L4, L5, L6 and L8, respectively. The secondary (post-cracking) slope was 10, 20, 30, 40 or 50% of the initial stiffness. As this is influenced primarily by amount of reinforcement, it was called R1, R2, R3, R4 and R5, respectively. A total of 13 different possible cases were considered.

A suite of 20 ground motions were selected from FEMA 440 study based on the spectrum in the long period range. The ground motions were used in two different ways: (i) the unmodified ground motions were scaled to give a ratio R of elastic demand to strength of wall ranging from 0.5 to 5.0, and (ii) the motions were modified first before scaling. The modified ground motions were created by spectral matching to the NBCC design spectrum for Vancouver, BC. The NBCC spectral acceleration values at 2 and 4 s were used, but the decrease between them was taken proportional to $1/T$ rather than linear. Beyond 6 s, the acceleration was assumed to vary proportional to $1/T^2$, and beyond 10 s the acceleration was taken constant.

The results for the case of modified ground motions, R = 5, and initial period of wall T_i = 3 s is shown in Fig. 2. Note the results shown are for the lowest effective stiffness values. The unmodified ground motions, smaller R values, or longer T_i values all results in larger effective stiffness values. Fig. 2 indicates that a lower-bound effective stiffness ratio $\alpha = E_c I_e / E_c I_g$ is 0.5 as suggested by FEMA 356. The upper-bound in Fig. 2 is 0.7; but much higher values are obtained with unmodified ground motions, smaller R values, or longer T_i values. Also shown in Fig. 2 are the predictions from the equal area under curve approach (for both the upper-bound and lower-bound force-displacement envelopes) and the secant stiffness to the yield point. Clearly, neither of these methods give a good estimate of effective stiffness because they do not account for the energy dissipated within the hysteresis loops.

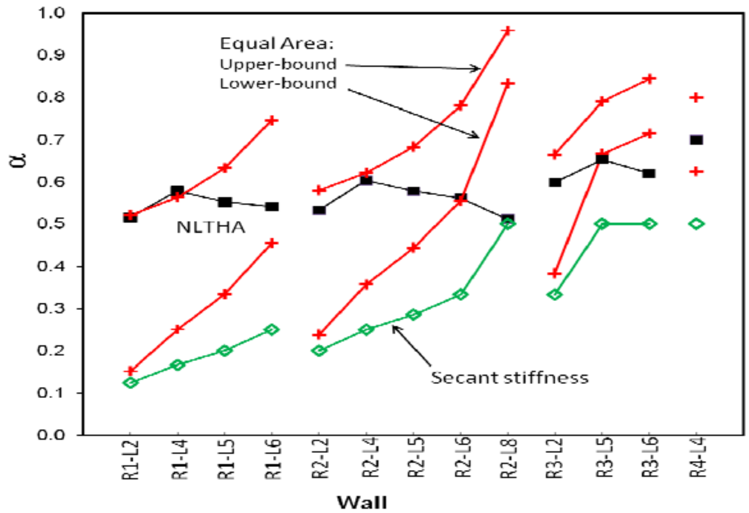

Fig. 2 Effective stiffness as a ratio of initial uncracked section stiffness for R = 5 and T_i = 3 s.

3. ROTATIONAL DEMANDS ON FLEXURAL WALLS

To ensure a wall has adequate flexural displacement capacity, inelastic rotational capacity of the wall θ_{ic} must be greater than the inelastic rotational demand θ_{id}. The inelastic rotational capacity of a wall is given by:

$$\theta_{ic} = (\phi_c - \phi_y)\, l_p \tag{1}$$

where the total curvature capacity ϕ_c equals the maximum compression strain of concrete ε_{cm} (typically taken between 0.003 and 0.005) divided by the compression strain depth c; the yield curvature ϕ_y can be safely estimated as $0.004/l_w$ for walls without confinement, and l_p is the length (height) that maximum inelastic curvature is assumed to be uniform (plastic hinge length). l_p has traditionally been assumed to vary between 0.5 and 1.0 times the wall length (horizontal dimension).

Experimental and analytical results (Bohl and Adebar, 2008) indicate inelastic curvatures actually vary linearly in walls; however the concept of maximum inelastic curvature over l_p can still be used to estimate flexural displacements of isolated walls. Based on the results of nonlinear finite element analyses using a model validated by test results, Bohl and Adebar (2008) presented the following expression for the lower-bound (safe) plastic hinge length in isolated walls:

$$l_p = (0.2 l_w + 0.05 z)\left(1 - \frac{1.5\,P}{f_c' A_g}\right) \le 0.8 l_w \tag{2}$$

where $z = M/V$ in the plastic hinge region of the wall; and a compressive axial load P is taken as positive. A typical value for z is $0.7h_w$, a typical value for h_w is $10l_w$, and a typical value for $P/f_c'A_g$ is 0.10. Substituting these values into Eq. (2) gives $l_p = 0.5l_w$, which is the typical value that is used as a safe lower-bound. When the ratio of z to h_w is smaller (due to higher modes), the height-to-length ratio of the wall h_w/l_w is smaller, and/or the axial compression stress is higher, the plastic hinge length is shorter.

To date there have been no recommendations for plastic hinge length of coupled walls. It is expected that walls with a low degree of coupling will act similar to separate cantilever walls. Thus, the wall length to be used in Eq. (2) for Partially Coupled Walls (degree of coupling < 67%) is the individual wall segment length. On the other hand, very highly coupled walls will act similar to a single solid wall with openings. In the absence of any better information, it is recommended that the wall length used in Eq. (2) for Coupled Walls (degree of coupling ≥ 67%) be the overall length of the coupled system.

The inelastic rotational demand on a concrete wall can be determined from:

$$\theta_{id} = \frac{\Delta_{id}}{\left(h_w - 0.5l_p\right)} \tag{3}$$

where Δ_{id} is the inelastic displacement demand, and $\left(h_w - 0.5l_p\right)$ is the effective height of the wall above the centre of the plastic hinge. While the lower-bound plastic hinge length from Eq. (2) gives safe results when estimating rotational capacity from Eq. (1), a longer plastic hinge length of $l_p = 1.0l_w$ gives safer results when estimating rotational demand from Eq. (3). Walls of different length that are tied together at numerous floor levels experience similar plastic rotations if they are subjected to the same top displacement (Adebar et al., 2005). Thus, one value of $\left(h_w - 0.5l_p\right)$ should be used for an entire system of walls acting together.

The remaining unknown in Eq. (3) is the inelastic displacement demand, which is the difference between the total displacement and the elastic displacement: $\Delta_{id} = \Delta_d - \Delta_y$. One approach is to assume that the elastic portion Δ_y is equal to the first mode yield displacement, which is a function of the wall height and length. This approach cannot easily be extended to coupled walls or systems with different length walls, as the yield displacement of these is not related to the dimensions of any individual wall. Another problem is that the first mode yield displacement increases exponentially with wall height; but the displacement demand of very tall walls will be limited by the maximum ground displacement. The solution to both of these is to determine the inelastic displacement as a portion of the total displacement demand of the seismic force resisting system. One method of doing this is to assume that the inelastic drift, which is equal to the inelastic rotation, is equal to the maximum global drift:

$$\theta_{id} = \frac{\Delta_{id}}{\left(h_w - 0.5l_p\right)} \approx \frac{\Delta_d}{h_w} \tag{4}$$

This approach was used to develop the wall ductility provisions in the 1999 ACI 318 building code (Wallace and Orakcal 2002). White and Adebar (2006) compared the inelastic rotations determined using Eq. (4) with the results of numerous nonlinear dynamic analyses on high-rise buildings, and found that this approach gives reasonable results for coupled walls; but may over predict the inelastic rotations in cantilever walls. The reason is that the elastic displacement $\Delta_y = \Delta_d - \Delta_{id}$ may be a larger portion of the total displacement than half the wall length ($0.5l_w$) is of the total wall height h_w in tall buildings.

Another possible approach is to relate the elastic portion of the total displacement to the relative strength of the wall. Unlike Eq. (4), such an approach would predict that a wall with adequate strength would not be subjected to any inelastic displacement demand, and would predict that an increasing portion of the total displacement is due to inelastic displacement as the strength is reduced. A simple rational expression for the inelastic portion of the total displacement demand is given by:

$$\theta_{id} = \frac{\Delta_d \left(1 - 1/R\right)}{h_w - 0.5l_w} \tag{5}$$

where R is equal to the ratio of elastic demand to strength of the wall. White and Adebar (2006) found good agreement between the predictions from Eq. (5) and the results from non-linear dynamic analyses of a variety of high-rise buildings with cantilever walls subjected to a wide variety of ground motions. They also found that Eq. (5) gives unsafe predictions for coupled walls, particularly if the walls have uniform coupling beam strengths over the height, thus Eq. (4) should be used for coupled walls.

4. COUPLING BEAM ROTATIONS

The Canadian code has traditionally required diagonal reinforcement in coupling beams with low span-to-depth ratios and high shear stress; but did not place any limits on inelastic demands put. When NLRHA was applied to a typical Canadian coupled wall system built in the US (Mutrie, Adebar and Leung 2000), coupling beam rotations were found to be critical and the geometry of the coupled walls had to be modified to meet rotation limits in FEMA 273. A procedure was developed by Adebar and White (2002) to estimate coupling beam rotations from RSA results, this procedure was implemented in the 2004 Canadian concrete code. The provision requires that the total rotational demand θ_d on coupling beams be less than the rotational capacity θ_c , which is 0.04 for coupling beams with diagonal reinforcement and 0.02 for beams with conventional reinforcement.

Adebar and White (2002) found that coupling beam rotations are proportional to the difference in wall slope and floor slope, where the latter is equal to the relative axial deformation of walls divided by the horizontal distance between the reference points. Using the wall centroids as the reference points gives satisfactory results that are usually safe compared to accounting for the shift in the neutral axis location in walls due to cracking of concrete and yielding of reinforcement. The wall slope associated with maximum coupling beam rotation, is proportional to maximum global drift, and is much greater than the critical floor slope. Thus, the level of maximum coupling beam rotation occurs near the location of maximum wall slope. This is usually in the lower levels of the coupled walls due to inelastic drift being uniform over the height, and coupling beams pulling back at the top of the walls. Due to the floor slopes, the maximum coupling beam rotations do not necessarily result from the maximum wall slopes during the earthquake.

A simplified procedure that gives reasonable results is to assume that the critical wall slope is equal to the maximum global drift, and the corresponding floor slope is equal to zero. This approach leads to the following equation for estimating the rotational demand on coupling beams which was incorporated into the 2004 Canadian concrete code:

$$\theta_{id} = \left(\frac{\Delta_d}{h_w}\right)\frac{\ell_{cg}}{\ell_u} \tag{6}$$

where l_{cg} is the horizontal distance between centroids of the walls on either side of the coupling beams, and l_u is the clear span of the coupling beam between the walls.

5. SEISMIC SHEAR DESIGN ISSUES

5.1 Shear Strength of Plastic Hinge Regions

Special design requirements are needed to ensure that a shear failure does not occur in the plastic hinge regions of concrete flexural walls. One requirement is that the maximum shear stress is reduced to account for damage from reverse cyclic inelastic rotation of the plastic hinge, which reduces the ability of concrete in that region to resist diagonal compression. Also, the quantity of transverse reinforcement needs to be increased to avoid accumulative yielding of transverse reinforcement. It is well known that the reduction in maximum shear stress and increase in transverse reinforcement in the plastic hinge region should be related to the rotational demands on the plastic hinge. Given that designers must already determine the inelastic rotation demands in order to evaluate the ductility (confinement) requirements, this parameter can easily be utilized in shear design.

In the procedure proposed by Adebar (2006) and implemented into the 2004 Canadian concrete code, the factored shear demand on plastic hinge region shall not exceed $0.10\,\phi_c f'_c b_w d_v$ unless the inelastic rotational demand on the wall θ_{id} is less than 0.015. When $\theta_{id} \leq 0.005$, the factored shear demand shall not exceed $0.15\,\phi_c f'_c b_w d_v$. For inelastic rotational demands between these limits, linear interpolation may be used. The effective shear depth d_v is equal to the internal flexural lever arm jd but need not be taken less than $0.8\ell_w$. The shear resistance of a wall shall be taken as:

$$V_r = V_c + V_s = \phi_c\,\beta\sqrt{f_c'}\,b_w d_v + \frac{\phi_s A_v f_y}{s}\,d_v \cot\theta \tag{7}$$

The value of β shall be taken as zero ($V_c = 0$) unless the inelastic rotational demand on the wall θ_{id} is less than 0.015. When $\theta_{id} \leq 0.005$, the value of β shall not be taken greater than 0.2. For inelastic rotational demands between these two limits, linear interpolation may be used. The value of the compression stress angle θ shall be taken as 45° unless the axial compression force acting on the wall is greater than $0.1\,f'_c A_g$. When the axial compression is greater than or equal to $0.2 f'_c A_g$, the value of θ shall not be taken less than 35°. For axial compressions between these limits, linear interpolation may be used.

5.2 Walls Supported by Stiff Diaphragms Below Flexural Hinge

High-rise concrete shear walls are often supported near or below grade by stiff floor diaphragms connected to perimeter foundation walls. When a large portion of the overturning moment in the wall is transferred to the foundation walls by two or more stiff floor diaphragms, the maximum bending moment (flexural plastic hinge) occurs above the diaphragms and the shear force reverses below the flexural hinge. Depending on the stiffness of floor diaphragms, and on the shear rigidity and flexural rigidity of the high-rise concrete walls, the reverse shear force below the flexural plastic hinge may be much larger than the base shear above the flexural hinge. This is sometimes referred to as the "backstay effect."

Nonlinear dynamic analyses (Rad and Adebar, 2008a) indicate the maximum reverse shear force is proportional to the bending moment capacity of the wall and inversely proportional to the accompanying base shear force. An upper-bound estimate of bending moment capacity of the high-rise wall combined with an assumed zero base shear force can be used in a simple nonlinear static analysis to estimate the maximum shear force below the flexural plastic hinge. A nonlinear shear model (Gerin and Adebar 2007) was used to determine whether diagonal cracking of the wall and yielding of horizontal wall reinforcement can reduce the reverse shear force without causing a shear failure. It was found that increasing the quantity of horizontal reinforcement in the wall will increase the maximum shear strain demand and may actually increase the chance of a shear failure in the wall if the amount of horizontal reinforcement is increased beyond a certain limit. An upper-bound estimate of floor diaphragm stiffness should be used in order to not underestimate the shear strain demand on high-rise walls. A detailed 11 step analysis/design procedure is given by Rad and Adebar (2008a).

6. DESIGN OF GRAVITY COLUMNS FOR SEISMIC DRIFTS

Requirements for members not considered part of the SFRS were first introduced in the 1994 edition of the Canadian concrete code and these were unchanged in the 2004 edition. The intent of these requirements is to ensure adequate deformability of all structural members that are subjected to seismically induced deformations. The code provides a simplified procedure that involves comparing the maximum bending moment induced in a column when the building is displaced laterally with the nominal bending resistance of the column. Unfortunately, designers in Canada have typically used the results from RSA to estimate bending moments in columns. While RSA can be used to estimate design displacement at top of walls, it does not give the correct displacement profile. Adebar (2005) found that the curvatures in a wall may be up to 5 times larger than estimated from RSA.

A simple design approach results from relating the maximum curvature demand in walls to the maximum curvature demand in gravity columns. Adebar (2005) proposed that these curvatures be assumed equal. Recent results from nonlinear finite element analysis of wall – column systems have indicated that the curvature demands in gravity columns may be much larger than in the wall if the column is fixed at the base (see Fig. 3). The curvature capacity of a section (wall or column) is equal to the maximum compression strain of concrete (typically taken between 0.003 and 0.005) divided by the compression strain depth. Thus the curvature

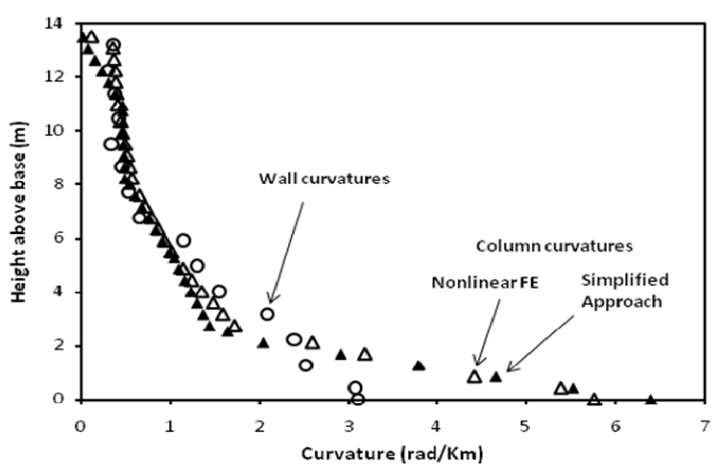

Fig. 3 Curvature demands in gravity column and core-wall.

capacity of a column can be increased by reducing the compression strain depth, which can be achieved by increasing the column width. The typical wall-columns (e.g., 12 in. wide by 48 in. long) that are not appropriate over the height of the plastic hinge region of the wall.

7. DYNAMIC MAGNIFICATION OF SHEAR

It is common practice to determine the design bending moments by reducing the elastic bending moments determined by RSA by up to a factor of 5 and to determine the design shear force by reducing the elastic shear force determined from RSA by the same reduction factor. NLRHA has shown that flexural yielding of a cantilever wall does not limit the shear force in the wall. The shear force tends to increase as the magnitude of ground motion is increased. This increase is often referred to as "dynamic shear amplification." When NLRHA is done to determine the dynamic shear amplification the wall is assumed to be elastic above the plastic hinge, and the uncracked shear rigidity is used in the analysis. Babak and Adebar (2008b) have found that flexural yielding of the wall at numerous locations over the height and reduced shear rigidity of the wall due to cracking greatly reduces the dynamic shear magnification.

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