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ON THE SEISMIC SHEAR DEMAND ON WALLS IN DUCTILE RC DUAL SYSTEMS

A. Rutenberg[1] and E. Nsieri[2]

ABSTRACT

Dual systems or wall-frames (WFs) are efficient lateral load resisting systems in tall buildings. Yet, although widely used in practice, critical aspects of their behaviour, mainly estimating post-yield seismic shear demand in the walls are not adequately covered by the provisions of many modern seismic codes, including leading ones, such as ASCE 7-05 and EC8-2004, as well as by some available approximate procedures. This is due to the fact that amplification of wall shear demand depends primarily on the fundamental natural period T, the strength reduction factor R or ductility demand, and the contribution of the walls to the total base shear η. The effects of these parameters on the shear demand amplification is the subject of the present study. A nonlinear dynamic parametric study is performed. WFs designed for realistic ranges of T, R and η are subjected to a suite of 10 ground motions. The results are compared with available formulas for predicting the wall shear forces, and particularly with the wall shear amplification formula given in the New Zealand concrete code (NZS3101-06) Commentary. As an alternative, a variant of the formula developed for cantilever wall structures is proposed.

Introduction

Dual systems or wall-frames (WFs, see Fig. 1) are known to be very efficient lateral load-resisting systems in tall buildings. Their efficiency is achieved by combining the advantages of their two constituent members: the relatively large stiffness of the walls limits storey drifts, and hence preventing storey mechanism instability; while the frames, through plastic hinges, usually at beam ends, often provide well distributed energy dissipation along the building height (e.g., Bertero 1984, Paulay and Priestley 1992).

The superior seismic response of these structures in the post elastic range has been recognized by building codes by assigning to them high strength reduction or behavior factors R (q in Europe), particularly when nonlinear analysis reveals their post-yield over-strength.

[1]Professor Emeritus, Faculty of Civil and Environmental Engineering, Technion, Haifa 32000, Israel, Email: avrut@tx.technion.ac.il
[2]Structural Engineer, Yaron-Offir Engineers Ltd, Technion, C.I.R building, Haifa 32000, Israel, Email: emad@yaron-offir.co.il

The linear response of WFs to horizontal loading has been extensively studied, particularly using the continuum approach (e.g., Stafford Smith and Coull 1991, Murashev et al. 1971), and their response is well understood. However, their post-yield behaviour is much less so, particularly the evaluation of shear demand on the walls.

Studies on shear demand amplification in WFs followed naturally from those on flexural walls. Blakeley et al. (1975) recognized that shear amplification is mainly due to the effect of higher vibration modes following the formation of plastic hinge at the wall base. Paulay and his coworkers (Goodsir et al. 1983) noted that, as in flexural walls, higher vibration modes amplified the shear force demand on the walls of WFs, but to a lesser extent, and proposed a simple prediction formula accounting for this response reduction. This extension of the shear amplification formula for flexural walls, based on Blakeley et al. (1975), to walls of dual systems is also given in Paulay and Priestley (1992).

Wall shear amplification in WFs was studied by Kabeyasawa (1987) who proposed an amplification formula depending on the number of storeys, and also by Aoyama (1987). Whereas Goodsir el at. (1983) related the wall shear amplification factor ω_v^* to the portion of the base shear carried by the walls η, Alwely (2004) made ω_v^* dependent on the non-dimensional frame-to-wall stiffness ratio αH as evaluated from the continuum approximation. In a recent paper Kappos and Antoniadis (2007) propose a procedure to estimate more realistically the shear demand in the upper storeys and also a modified version of the Goodsir et al. (1983) formula (Eq. 1) to account for shear amplification in WFs with unequal walls.

The codified guidance regarding the amplification factor ω_v^* is rather limited. The New Zealand seismic code NZS 3101 (2006) only makes a reference to Paulay and Priestley (1992) where Eq.1 is presented. This factor is made dependent on the number of storeys *n* and on η, as follows:

$$\omega_v^* = 1 + (\omega_v - 1)\eta \quad (1)$$

$$\omega_v = \begin{cases} 0.9 + \dfrac{n}{10}, & n \le 6 \\ 1.3 + \dfrac{n}{30} \le 1.8, & n > 6 \end{cases} \quad (2)$$

where ω_v is the dynamic base shear amplification factor for cantilever wall systems as given in Paulay and Priestley (1992). It is clearly seen that when $\eta \le 1.0$, $1 \le \omega_v^* \le \omega_v$.

Eq. 1 assumes that ω_v^*, as ω_v, is independent of the expected ductility demand or on R. Note also that Eq. 2 depends indirectly on the fundamental period T through its dependence on *n*.

From Eurocode 8 (EC8, CEN 2004) it may be concluded that in high ductility class systems when the wall share of the total WF base shear is more than 50%, the flexural wall shear amplification should govern. The EC8 amplification factors for the medium- and high-ductility classes (DC-M and DC-H respectively) vs. T and R are plotted elsewhere (see, e.g., Rutenberg and Nsieri 2006). Applicable US codes do not include shear amplification requirements even for flexural walls (except the Commentary to the 1999 SEAOC code). Quite recently Priestley et al. (2007), in the draft displacement-based seismic design code at the end of their book, proposed a dynamic base shear amplification expression for walls of WFs given in Eq. 3, which requires predicting the displacement ductility demand μ_{sys} of the dual system. When $0.4 \le \eta \le 0.8$ the wall base shear amplification is given by:

$$\omega_v^* = 1 + \frac{\mu_{sys}}{\varphi} C \qquad (3)$$
$$C = 0.4 + 0.2(T - 0.5) \le 1.15$$

where φ is an overstrength factor usually equals to 1.25. This formula is presented graphically in Fig. 5. Note that it assumes that ω_v^* is independent of η, hence cannot be in agreement with Eq. 1. As can be seen from the above, despite the well known advantages of dual systems and their widespread use, estimating the post-yield seismic shear demand amplification in the walls, important for precluding non-ductile failure, remains uncertain. Shear amplification in WFs is ignored (US codes), or rather crudely addressed (EC8), and when a procedure is provided, the two available ones (NZS 3101; Priestley et al. 2007) do not appear to be compatible.

In view of this rather unsatisfactory state of affairs a parameter study was carried out to evaluate ω_v^* values for several WF configurations with a view to providing additional data. The results were then compared with two realizations of Eq.1, and with Eq. 3. An alternative formula is proposed which better predicts the shear amplification for the data used for the present study.

General Description of the Studied Buildings, Modelling Assumptions and Loading History

The floor plans of idealized hypothetical 10, 15 and 20 storey symmetric concrete buildings are shown in Figs. 2, 3 and 4 respectively. For the present study only the y-direction was considered. A 3.5m storey height was taken for the 10 storey building, and 3.65m for the 15 and 20 storey buildings. As can be seen, each structure consists of 5 internal 3-bay frames and a flexural walls located at each opposite end of the floor plan. For simplicity, the reinforced concrete walls and frame components were kept constant throughout the height of the buildings. RC floor thicknesses were taken as 0.17m, 0.20m and 0.25m respectively for the 10, 15 and 20 storey buildings. These floor thicknesses were considered in evaluating the flexural stiffness of the T beams. Full fixity at base level was assumed, although it is often not realistic, particularly for the walls. The following effective moments of inertia I_{eff} were assumed: for T beams $0.4I_g$, columns $0.5I_g$ and walls $0.3I_g$ (I_g is the gross moment of inertia). The shear area A_s was conventionally taken as $= 5/6\,A_g$ (A_g is the gross rectangular cross-sectional area). Material properties were taken as follows: for concrete $E_c = 30\text{GPa}$, $G_c = 12\text{GPa}$, $f_c' = 40\text{MPa}$, and for steel reinforcement $E_s = 200\text{GPa}$, $f_y = 400\text{MPa}$.

These buildings were designed using linear code-based static equivalent lateral force (ELF) procedure in which the base shear was triangularly distributed along the building height. Gravity loads were included in the analysis. For simplicity, the reinforcement of columns and walls was taken as uniform along the height, while girder reinforcement was somewhat varied to approximately follow the static results.

The design parameters varied in the study were: the elastic wall base shear to total base shear ratio $\eta = V_w/(V_w + V_f) = 0.65, 0.8, 0.9$ (V_w, V_f are wall and frame shear at base), and strength reduction factor R = 1, 3, 5. Fundamental periods of the WF structures used in the parameter study are given in Table 1.

In order to obtain the chosen η values and the following fundamental natural periods: 1.0sec, and 1.5sec for the 10-storey, 2.0sec for the 15-storey, 2.5sec, and 3.0sec for 20-storey the storey masses of the buildings and the stiffness of the walls were adjusted. However, one modelling problem appeared to be ensuring a uniform basis for R=1 for all the buildings, i.e., that at the wall yield threshold none or only very few of the frame members actually yield. In many cases this required either relatively weaker walls or stronger beams than required on the basis of the ELF analysis. Also, no allowance was made for within-span plastic hinges; hence it was necessary to design for higher than required positive beam moment capacities.

The ground motion input for the parametric study consisted of the first 25 seconds of 10 records from the SAC project Los Angeles suite as given in Table 2. These suites have an exceedance probability of 10% in 50 years or a return period of 475 years (Somerville et al. 1997). The ground motion for each record was first factored so as to lead to imminent wall yielding, and this was taken as R =1; for other R values the ground motion was factored by the desired R, herein 3 and 5. As already noted, this often required some strength adjustments in the structural elements.

The 2-dimensional version of the computer code RUAUMOKO (Carr 2000) was used in the analysis assuming 5% tangent stiffness Rayleigh damping in the 1st and 5th modes. Axial load bending moment interaction diagram was accounted for the frame columns using the concrete beam-column yield surface capability in Carr (2000). The inelastic behaviour of the beams and walls was modelled as a one-component Giberson beam-element (Carr 2000). RC elements were assumed to behave as elastic-perfectly plastic material, and plastic hinges could form only at the bases of walls and columns and at both ends of beams. In the analyses the 5 frames were lumped into a single "super-frame" and the 2 walls into a "super-wall" having the combined properties of stiffness and strength. It was also assumed that the floor slabs are rigid in their own plane and completely flexible out of plane. In-plane floor rigidity was assumed and modelled by slaving the lateral displacements of adjacent nodes to a master node. Altogether circa 450 computer runs were carried out.

Analyses and Results

Results of the parameter study are given in Figs. 5 and 6, showing the variations of the mean amplification factor $\omega_v^* = V_{aw}/V_{dw}$ with T for several values of R and η for the limited Los Angeles SAC suite. Note that V_{aw} is the mean value of the dynamic wall base shear as obtained from the parametric study, and V_{dw} is the wall base shear as evaluated by means of the static ELF procedure. As can be seen in Fig. 5 or 6, the extent of wall base shear amplification depends on three main parameters: (1) fundamental period T of the dual system, (2) strength reduction factor R approximating the expected ductility demand, (3) the walls base shear ratio η. It is also seen that the larger the fundamental period T the more important is the effect of higher modes on the response, mainly after the formation of a plastic hinge at the wall base, amplifying wall base shear with respect to the linear static one as obtained from the ELF procedure. The effect of η appears to be less pronounced.

Comparing the results of the parametric study with those obtained using Goodsir's and Priestley's formulas requires making some assumptions regarding the parameters appearing in them, mainly the relation of the displacement ductility demand μ_{sys} of the WF system to R.

Results based on Eq. 1, assuming that in Eq. 2 the relation of T to the number of storeys *n* is given by the code formula $T = 0.05H^{0.75}$ (H is the building height), are also shown in Fig. 5. By comparing the results it is evident that Eq. 1 with T computed from *n* per the code formula underestimates the base shear demand amplification at medium and high R levels. Results based on Eq. 3 for $\eta = 0.65, 0.80$, and assuming the simplest relationship $\mu_{sys} \cong R$ (according to the equal displacement assumption), are also shown in Fig. 5.

Comparing Eq. 3 with the parametric results it is seen that this formula overestimates the base shear amplification demand. Hence for the structures used in the present study results based on Eq. 3 appear to be very conservative. However, such comparison is perhaps not legitimate since the natural periods in Eq. 3 are ductility dependent; namely, increasing μ_{sys} results in increased T, whereas in the present study they are not R dependent.

Results are also shown in Fig. 5 when ω_v is evaluated from Eq. 4 (Rutenberg & Nsieri 2006), rather than Eq. 2, and substituted into Eq. 1 to obtain ω_v^* as follows:

$$\omega_v = 0.75 + 0.22(T + R + TR) \qquad (4)$$

$$\omega_v^* = 1 + (\omega_v - 1)\eta = 1 + [-0.25 + 0.22(T + R + TR)]\eta \qquad (5)$$

However, this substitution gives unsatisfactory agreement with the computed results. Yet, since it appears that the amplification is roughly linear in T, R and η, the following formula is proposed for walls in dual systems comprising flexural walls of equal lengths:

$$\omega_v^* = 0.4 + 0.2(T + R + \eta) + 0.13TR\eta \qquad (6)$$

The parametric results are compared in Fig. 6 with the proposed formula. The agreement appears satisfactory with some exceptions. However, the formula should be considered as tentative pending checking for inconsistencies in the designs and further study, particularly in view of the discontinuity in response between $\eta = 0.9$ and $\eta = 1.0$ (Eq. 4). Note that the values of ω_v^* do not include a base moment overstrength factor required in several seismic codes. Also note that when modal analysis (rather than the ELF procedure) is carried out, it already provides the correct linear elastic (R=1) shear amplification.

Summary, Discussion and Conclusions

This paper is concerned with the seismic shear force demand on walls in wall-frame (WF) or dual systems serving as the lateral load resisting system in multistorey buildings. The main factor affecting the shear demand amplification on the walls is the effect of higher modes of vibration mainly in the post-yield range.

It has been shown by parametric analysis that for the walls studied herein: (1) the wall base shear amplification factor Eqs. 1 and 2 (Goodsir et al. 1983), which the New Zealand seismic code NZS 3101 (2006) refers to, are not conservative, particularly WFs designed for large ductility demands (large strength reduction factors); (2) the EC8 (CEN 1998-1, 2004) shear amplification formula is in need of calibration, since it does not adequately predict the expected response for DC-H structures, and is non-conservative for DC-M ones; (3) the base shear amplification expression for the walls Eq. 3 (Priestley et al. 2007) overestimates shear amplification demand when assuming the simplest relationship: $\mu_{sys} \cong R$. Equation 6 could serve as an alternative to available formulas for WFs with equal walls, but for code purposes a larger data base appears to be required.

Studies on shear demand in flexural wall systems comprising unequal walls (Rutenberg and Nsieri 2006) and on similar WF systems (Kappos and Antoniadis 2007), have shown that the relative contribution to the base shear of each wall may not be satisfactorily predicted by its relative flexural strength, as is usually the case. This is due to the redistribution of shear forces from the longer walls in which plastic hinges form first to the shorter ones. Simple reasoning and preliminary analyses suggest that in WFs this effect is not likely to be significant when η - the base shear portion carried by the walls - is not dominant. Further studies are now under way.

Large residual deformations and internal forces have been observed in the parameter study results (for R=5). This issue is beyond the scope of the paper, but needs to be addressed.

The behaviour of real systems is, of course, much more complex than assumed herein. First is the elastic-plastic model used in this study. The limited study carried out with the Takeda model, which is more realistic for reinforced concrete members, predicts smaller shear demand amplification (less conservative). Other modelling limitations include: full wall fixity at foundation level is an idealization (unaccounted for rocking and strain penetration); shear stiffness falls when diagonal cracks form; and some shear deformation ductility is available. Also, the floors are not infinitely rigid in their own planes. These effects may modify the shear distribution between the walls and frames, and lower the amplification.

Although several factors mitigate the extent of shear demand amplification, adapting design procedures and seismic code provisions to reflect this effect more faithfully is called for.

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Table 1. Number of storeys and fundamental periods of WF buildings used in study

Fundamental period (seconds)	**Number of storeys**		
	10	15	20
T	1.0 , 1.5	2.0	2.5 , 3.0

Table 2. Ten LA records used in study (10% in 50 years probability of exceedance)

Designation	**Record**
LA02	Imperial Valley, 1940, El Centro
LA04	Imperial Valley, 1979, Array #05
LA06	Imperial Valley, 1979, Array #06
LA08	Landers, 1992, Barstow
LA10	Landers, 1992, Yermo
LA12	Loma Prieta, 1989, Gilroy
LA14	Northridge, 1994, Newhall
LA15	Northridge, 1994, Rinaldi RS
LA18	Northridge, 1994, Sylmar
LA19	North Palm Springs, 1986

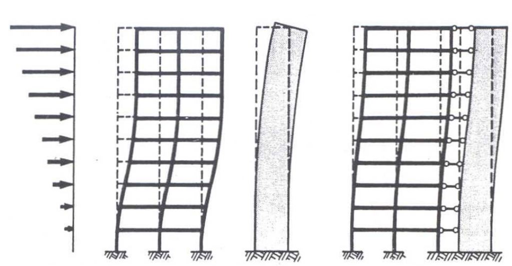


Figure 1. Lateral deformation shape under horizontal seismic forces of a frame, a wall, and a dual system (Paulay and Priestley 1992)

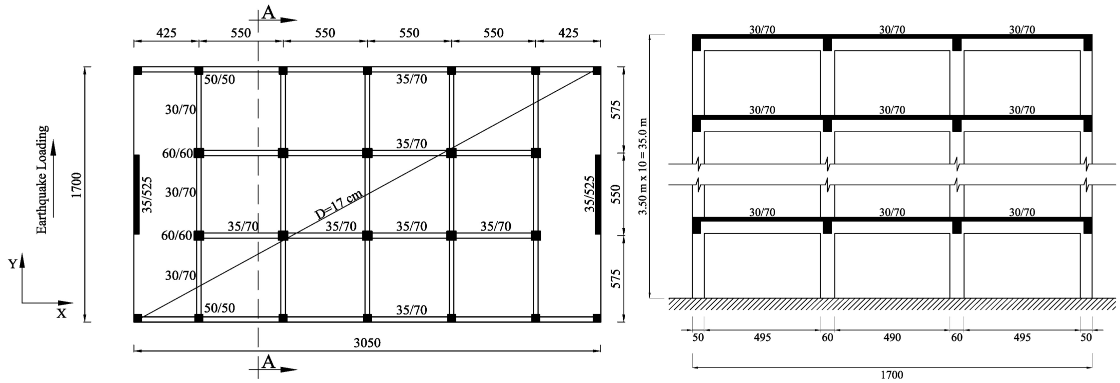


Figure 2. Plan and section A-A of the studied 10-storey RC building (η = 0.8)

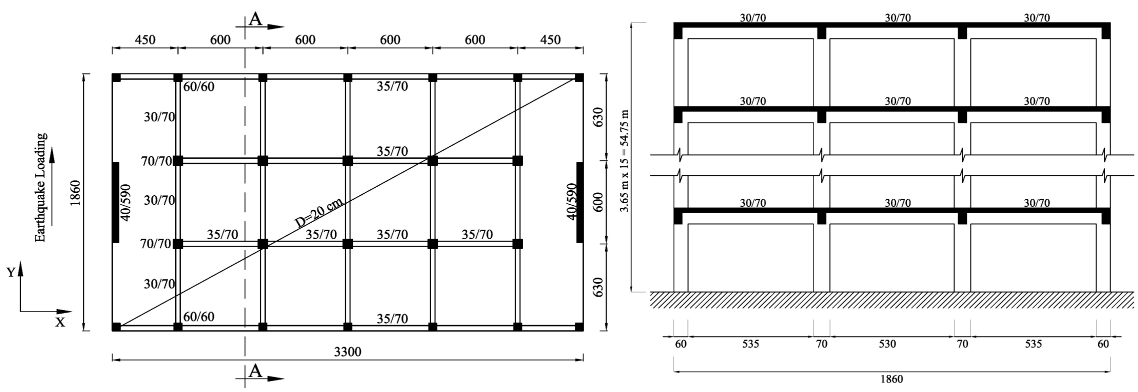


Figure 3. Plan and section A-A of the studied 15-storey RC building (η = 0.8)

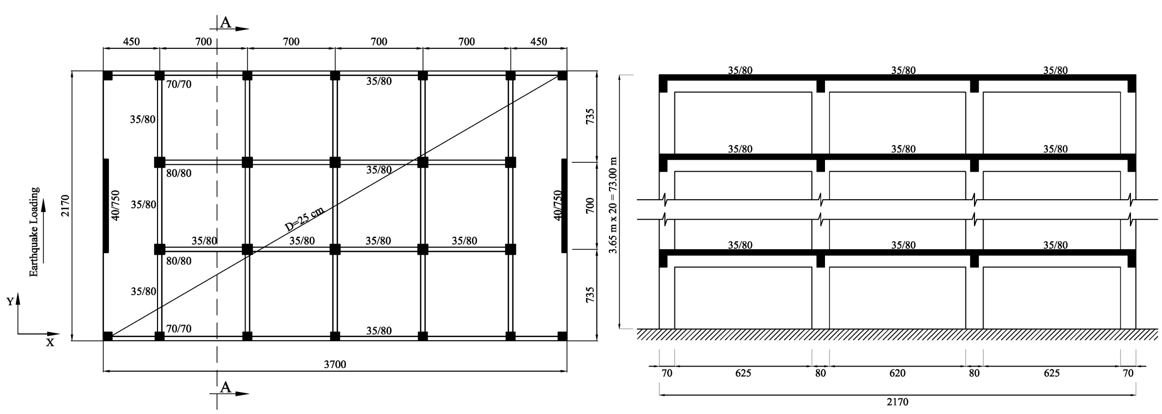


Figure 4. Plan and section A-A of the studied 20-storey RC building (η = 0.8)

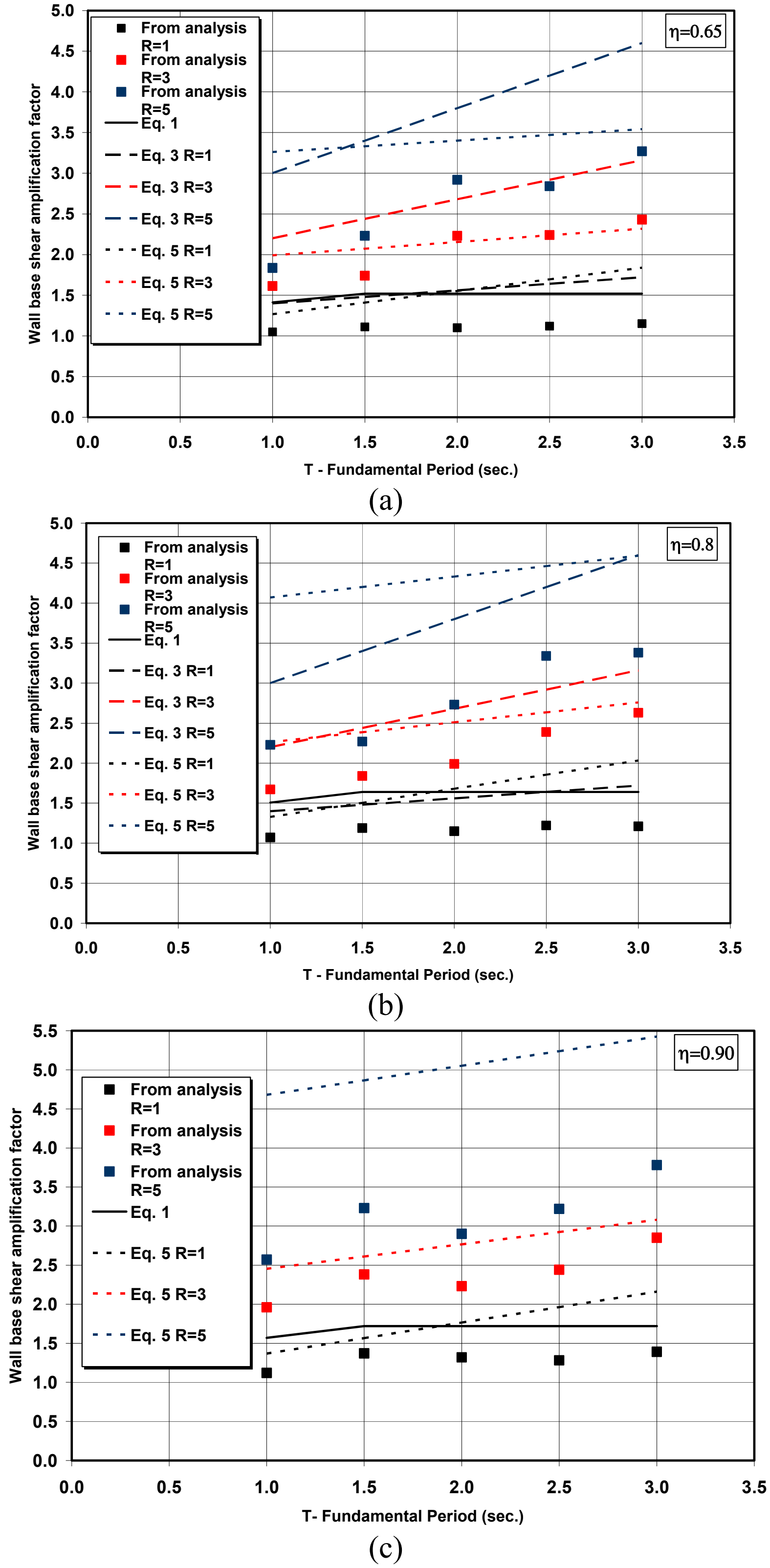
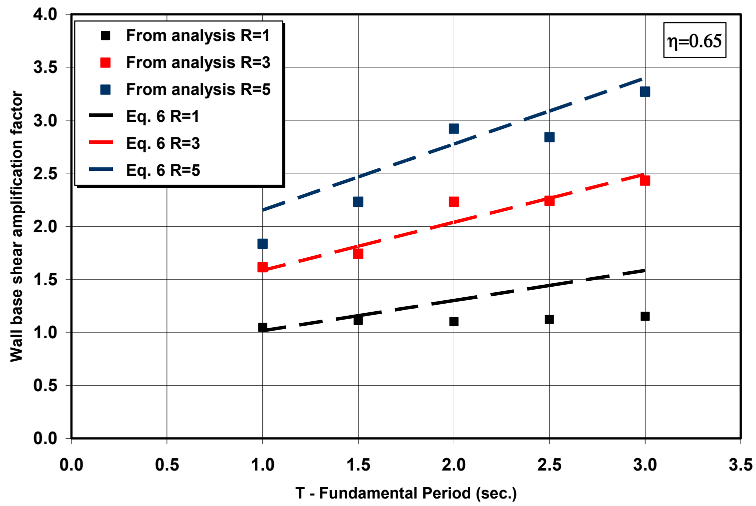
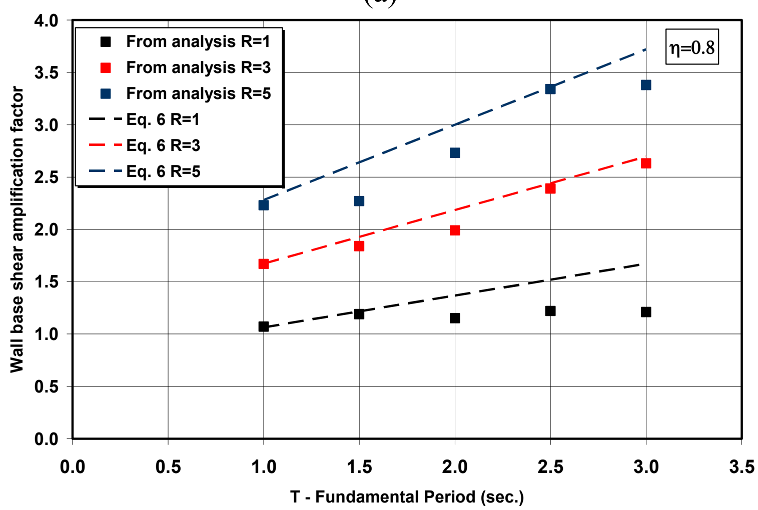


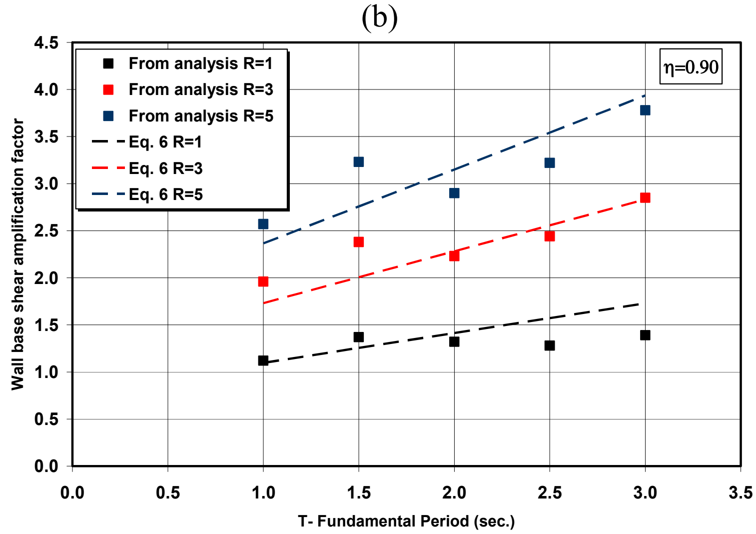
Figure 5. Mean results from parametric study, Goodsir et al. 1983 (Eq. 1), Priestley et al. 2007 (Eq. 3), and Eq. 5: (a) $\eta = 0.65$; (b) $\eta = 0.8$; (c) $\eta = 0.9$



(a)



(b)



(c)

Figure 6. Mean results from parametric study and proposed formula (Eq. 6): (a) $\eta = 0.65$; (b) $\eta = 0.8$; (c) $\eta = 0.9$