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EFFECT OF CONFINEMENT OF FLANGED WALL–SLAB JOINT UNDER LATERAL CYCLIC LOAD

S. Greeshma*, L. Annilet Sheeja and K.P. Jaya
Division of Structural Engineering, Department of Civil Engineering, College of Engineering Guindy, Anna University, Chennai – 600025, India

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ABSTRACT

An analytical study has been made of the interaction between laterally loaded flanged shear wall and floor slabs in building with cross- shear wall. Particular attention has been paid to study the influence of Height of Shear Wall (H) and Effective width of the slab (We) on the performance of wall – slab joint in a multi-storey building with shear wall. To carry out the analytical investigations, the structure was modelled in a Finite Element software ANSYS (Version 10 [1]). The specimens were sorted into two groups (Type 1 and Type 2) based on the ratio of height of shear wall and the effective width of the slab (H/We). The joints are detailed as per the provisions given for beam – column joint in IS 13920 [2]. The models were subjected to displacement-controlled lateral cyclic loading applied at the slab end. The performance of the connections in terms of the Von Mises stress, load-displacement hysteretic behaviour, ultimate load and energy dissipation were compared. Type 2 model performed better when compared to Type 1 model in terms of strength and energy dissipation.

Keywords: Effective width, wall–slab joint, cyclic load, detailing

1. INTRODUCTION

Shear walls are specially designed structural walls incorporated in buildings to resist lateral forces that are produced in the plane of the wall due to wind, earthquake and other forces. The most important property of shear walls for seismic design is that it should have good ductility under reversible and repeated over loads. In planning shear walls, we should try as much gravity forces as it can safely take. They should be also laid symmetrically to avoid torsion stresses. The forces are distributed to the shear wall of the building by the

* E-mail address of the corresponding author: greeshmas@annauniv.edu (S. Greeshma)

diaphragms and the shear wall transmits the loads down to the next lower storey or foundation. Diaphragm is a nearly horizontal structural unit that acts as a deep beam or girder when flexible relative to the support and as a plate when its stiffness is higher than the associated stiffness of the wall.

During an earthquake, it is the destruction of buildings and structures which mainly causes loss of lives. The vast extent of damage and the consequent loss of life associated with earthquakes reflect the poor construction practice in India. Existing multistoried buildings in earthquake prone regions of India are vulnerable to severe damage under earthquakes as proved by the Bhuj earthquake January 26, 2001. The structures which are less earthquake resistant succumb during an earthquake and add more to the damage. In order to build earthquake resistant structures, considerable research and dissemination of information is necessary in the design, detailing and performance of earthquake resistant structural elements.

2. RESEARCH SIGNIFICANCE AND REVIEW

Slab-wall connections in structure resisting lateral forces constitute a potential weak link in the preferred load path from slabs to walls, thereby influencing the pattern of lateral load distribution to the vertical members of system. One of the most critical areas in the design and construction of seismic resistant structures is the shear wall-slab joint. Structural responses during recent earthquakes (the Northridge earthquake January 17, 1994, the Bhuj earthquake January 26, 2001) indicate that joint failures are caused mainly by inadequate ductility due to improper connection between the shear wall and the floor slab.

The junction between the wall and the slab is subjected to severe stress concentration. This problem has already been studied theoretically by Coull and Wong [3] to determine the distribution of shear stresses at the junction. Bhatt et al. [4] produced charts for determination of design moments in coupling slabs due to lateral loads. He tested the RCC models with vertical bars around the wall periphery as shear reinforcement. Pantazopoulou et al. [5] presented a method for the connections between floor slabs and shear walls and their performance can influence the pattern and distribution of lateral forces among the vertical elements of a structure. Hossain [6] presented a paper on non-linear performance of slabs in coupled shear wall structures. The non-linear coupling action of reinforced concrete slabs in shear wall structures is investigated by the finite element method and the results are validated by small scale model tests using micro-concrete. Memon and Narwani [7] presented the results of experimental behaviour of connecting beams in a laterally loaded shear wall building based on the results of first two models of a tall building tested by them. Vinoth et al. [8] presented results of experimental behaviour of wall-slab joint in a laterally loaded shear wall building. Greeshma et al. [9] studied the response of shear wall – floor slab connection containing various types of shear reinforcement when subjected to gravity and lateral cyclic loading. The two types of specimens were $1/4^{th}$ scale representations of exterior shear wall – floor slab connections in a prototype RC building. The controlled specimen had no shear reinforcement and is failed due to shear. But the specimens with slab shear reinforcement proved equally effective in resisting the shear failure.

3. ANALYTICAL MODEL

A six storied R.C. building with exterior flanged shear wall (12 m x 7.5 m in plan) is modeled, analyzed and designed using ETABS software [10] and the shear forces, bending moments and axial forces around the wall-diaphragm interface due to different load combinations were obtained. Seismic analysis was performed using equivalent lateral force method given in the Indian Standard Code IS 1893:2002 [11]. One of the exterior shear wall and the slab was designed and detailed as per the design criteria of IS 456:2000 [12] and IS 1893:2002 [11] incorporation of the ductile detailing as per IS 13920:1993 [2]. The analytical model in ETABS is shown in Figure 1.

Having designed the structure, one of the shear wall – slab joint (marked 'A' in Figure 2) is subjected to finite element analysis. A scale factor of four has been adopted for modeling due to the convergence problems. Thus the original structure has been reduced four times following the laws of similitude. Following the laws of similitude, the reinforcements are also reduced to 1/4th of the design areas of the reinforcements of the prototype as shown in Table 1.

Table 1: Details of reinforcement in shear wall and slab

<table>
<tr><td rowspan="3">Shear wall</td><td>Vertical bars</td><td>24- 24 Φ bars (2 layers)</td></tr>
<tr><td>Horizontal bars</td><td>16 Φ bars @ 300 c/c</td></tr>
<tr><td>Stirrups</td><td>8 Φ bars @ 300 c/c</td></tr>
<tr><td rowspan="2">Slab</td><td>Longitudinal bars – Along the direction of shear wall.</td><td>12 Φ bars @ 200 c/c at the two ends of the slab and 12 Φ bars @ 300 c/c for the remaining portion.</td></tr>
<tr><td>Transverse bars – Perpendicular to the direction of shear wall.</td><td>12 Φ bars @ 175 c/c at the two ends of the slab and 12 Φ bars @ 300 c/c for the remaining portion.</td></tr>
</table>

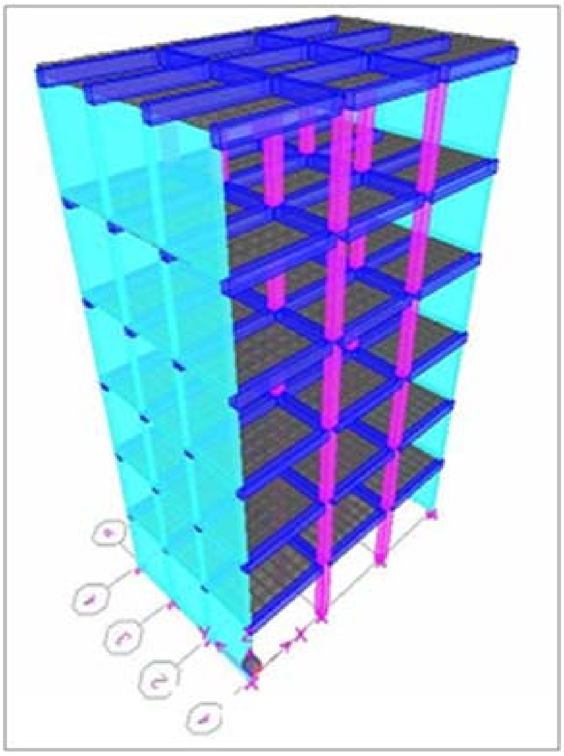

Figure 1. Analytical Model in ETABS

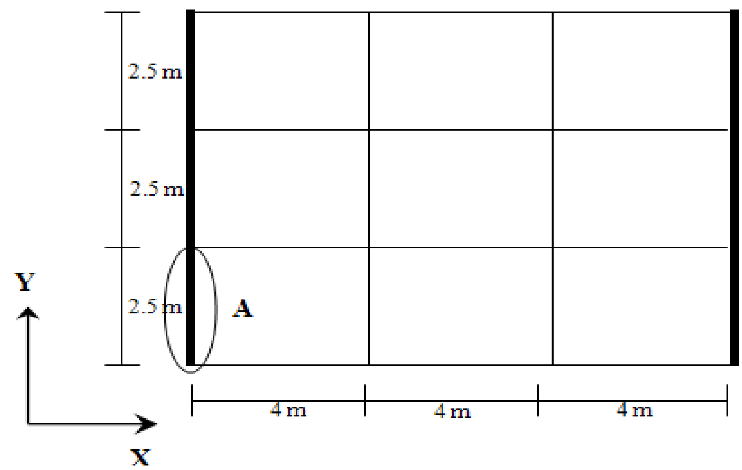

Figure 2. Plan of the prototype building

3. SPECIMENS AND VARIABLES

The analytical investigations included two types of models to study the effect of various influential parameters such as height of shear wall (H) and effective width of slab (W_e). The normalized parameter considered as 'the ratio of height of shear wall to effective width of the slab (H/W_e)'. The detailing adopted for the model is with 90° degree bent slab bars at the shear wall-slab joint (Greeshma et al. [9] with additional U hooks of 3 mm Φ connecting the shear wall and extending to the slab for the effective width of the slab (W_e). The specimens were designated as Type 1 (H/W_e =2.8) and Type 2 (H/W_e =2.25) based on the ratio of height of shear wall (H) to effective slab width (W_e). The reinforcement details for the both types of scale models are tabulated in Table 2.

Table 2: Reinforcement details of shear wall, slab and joint

<table>
<tr><th>Item</th><th colspan="2">Reinforcement details</th><th>Remarks</th></tr>
<tr><td rowspan="3">Shear Wall</td><td>Vertical Reinforcement</td><td>24 Φ6</td><td rowspan="5">Detailing with the provision of U bars extending to the slab for a length of 312.5 mm for Type1 specimens.

Detailing with the provision of U bars extending to the slab for a length of 388.5 mm for Type 2 specimens.</td></tr>
<tr><td>Horizontal Reinforcement</td><td>Φ6 @ 60 c/c for a distance of 167 mm at either side of joint and 120 mm c/c for remaining portion</td></tr>
<tr><td>Stirrups</td><td>Φ3 @ 60 c/c through out the entire length of the shear wall for a distance of 167 mm from either side of joint and Φ3 @ 120 mm c/c is restricted within the boundary length of the shear wall for remaining portion.</td></tr>
<tr><td rowspan="2">Slab</td><td>Longitudinal Reinforcement</td><td>Φ6 @ 62 mm c/c for a distance of 70 mm from the slab end and 90 mm c/c for remaining length.</td></tr>
<tr><td>Transverse Reinforcement</td><td>Φ6 @ 54 mm c/c for a distance of 61.5 mm from the slab end and 100 mm c/c for remaining length.</td></tr>
<tr><td>Joint</td><td>Transverse Reinforcement</td><td>Φ3 @ 30 mm c/c stirrups at the joint.</td><td>Confining reinforcement at joint as per IS 13920:1993 (provision for beam -column joint is extended for wall-slab joint).</td></tr>
</table>

2.1 Parameters Varied

The parameters considered for the study are height of shear wall (H) and effective width of the slab (W_e). Height of shear wall is considered to be same for all the specimens and is 875 mm for $1/4^{th}$ scale model. The effective width of the slab (W_e) is the width of the slab adjacent to the shear wall that is used to resist the collector forces. The procedures outlined in "Seismology and Standards Committee [13]" propose two methods to determine the effective slab width.

1- Effective slab width to resist the collector forces based on assumed 45 degree influence line, which originates from the "point of zero force" along the collector force diagram shown in Figure 3.

$$W_e = \frac{L_w}{2} \tan\theta = \frac{625}{2} \tan 45°$$

Slab Effective width, $W_e = 312.5$ mm

The ratio, $H/W_e = 2.8$

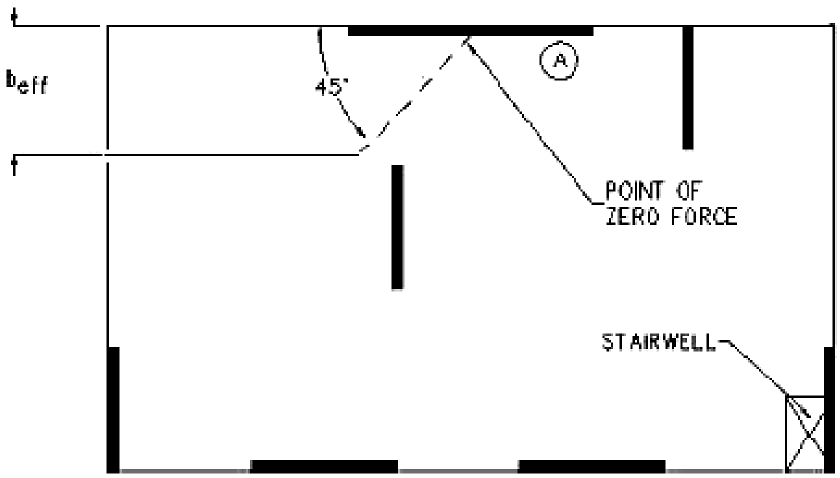

Figure 3 Effective slab width as collector

2- Effective slab width to resist the collector forces is determined arbitrarily from the following equation

$$W_e = t_w + n \ \frac{L_w}{2}$$

Where, n is the number of sides that slab exists adjacent to shear wall.

Slab effective width, $W_e = 75 + \dfrac{625}{2} = 387.5mm$. Hence, $H/W_e = 2.25$.

4. FINITE ELEMENT MODELING OF WALL – SLAB JOINT

For modeling of shear wall – slab joint, ANSYS Multiphysics (Version 10) [1] was used. The elements used were SOLID 65 for concrete and LINK 8 for reinforcement modeling. SOLID 65 elements have eight nodes with three degrees of freedom at each node- translations in the nodal x, y and z directions. Link 8 element, three- dimensional spar element has two nodes with three degrees of freedom at each node – translations in the nodal x, y and z directions.

4.1 Sectional Properties

Since there was no smeared reinforcement, the real constants for Solid 65 element (volume ratio and orientation angle) were set to zero. The real constants correspond to Link 8 elements are cross sectional area and initial strain. The sectional properties adopted for the model are described in Table 3.

Table 3: Real constants for steel reinforcement (Link 8 element)

Designation of specimen	Real Constant Set	Element Type	Particulars of the Specimen	
Type 1 and Type 2	2 & 3	Vertical & Horizontal reinforcement of shear wall	Cross sectional Area (m^2)	$28.27x10^{-6}$
			Initial Strain	0
	4	Shear reinforcement of shear wall	Cross sectional Area (m^2)	$7.07\ x10^{-6}$
			Initial Strain	0
	5	Longitudinal and Transverse reinforcement of Slab	Cross sectional Area (m^2)	$28.27x10^{-6}$
			Initial Strain	0
	6	Additional U bars connecting diaphragm to shear wall	Cross sectional Area (m^2)	$7.07\ x10^{-6}$
			Initial Strain	0

4.2 Material Properties

For the reinforcing bars, the yield stress was obtained from testing as f_y = 432 MPa and the tangent modulus as 847 MPa. The average 28-day cube strength (f_{cu}) of tested specimens was 44.22 MPa. The cylinder strength ($f_c^{'}$) is adopted as per ACI Code [14] and thus the ultimate compressive strength ($f_c^{'}$) was 35.376 MPa. The shear transfer coefficients for modeling the wall – slab joint are adopted according to Wolanski [15]. The equation developed by Desai and Krishnan [16] was used for the stress-strain relation for concrete.

4.3 Boundary Conditions

In order to fix the base of the shear wall, all nodal dofs are constrained at the base of the wall. At the end of the end, all degrees of freedom are constrained except in-plane displacement and rotations θ_x and θ_z. The shear wall – slab joint is subjected to varying lateral load (reversible cyclic load) at the end of the slab. The axial load and the moment at the mid section of the second storey are distributed to the nodes along the length of the shear wall as shown in Figure 4.

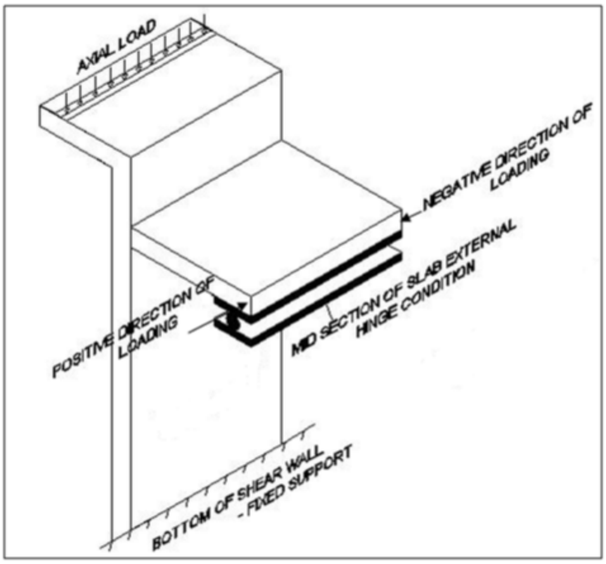

Figure 4. Model showing the support conditions and loading

4.4 Loading Conditions

The loading is controlled by drift ratio for both the models, where the drift ratio is defined as the deflection of the load point divided by the distance between the load point and the centre line of the shear wall. The models were analyzed with reversible cyclic loadings in the positive (pushing the slab along the length of shear wall) and negative direction (pulling the slab along the length of shear wall).

4.5 Finite element Model

The shear wall – slab joint is modeled in ANSYS and the modeling details are shown in Figure 5 (a) to (b).

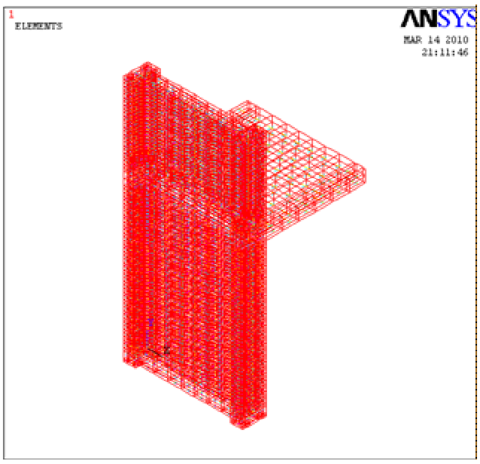

Figure 5. (a) Reinforcement configuration of wall – slab joint

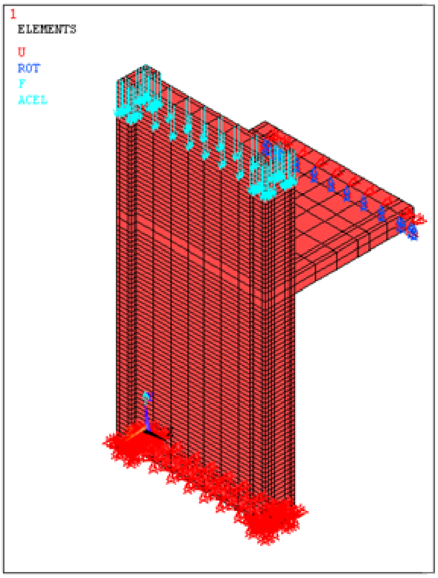

Figure 5. (b) Wall – slab joint model with boundary conditions and loading

5. FINITE ELEMENT ANALYSIS

The finite element analysis has been carried out for the shear wall- slab joint subjected to cyclic lateral drift histories. The displacement convergence criterion was used with the tolerance of 0.001. The command prompt line input data was adopted for applying the reversible cyclic loading. The analysis type has been mentioned as transient.

6. RESULTS AND DISCUSSIONS

Finite element analysis is carried out for both the types of models. The results of the analysis are presented as follows.

6.1 Ultimate Load

The variation in ultimate load for the two categories of detailing for cyclic loading is shown in Table 4. It can be observed that the ultimate strength increased for Type 2 detailing of reinforcement compared to that of Type 1. The change is nearly 12.3%.

Table 4: Ultimate load of specimens

Designation of Specimen	Ultimate Load (kN)		
	Positive Direction	Negative Direction	Average (P_u)
Type 1	24.520	23.152	23.836
Type 2	26.862	26.675	26.768

The increase in ultimate load for Type 2 model can be attributed due to the presence of reinforcement in the critical sections (for higher effective width of H/2.25) which experience higher shear and flexural stresses due to cyclic loading.

6.2 Load – Drift Hysteretic loops

The hysteretic behavior of Type 1 model shows a moderate level of pinching, which may be attributed to inadequate confinement in the connection region. In addition, the Type 1 specimen showed a sudden decrease in strength. However Type 2 specimens exhibited large, stable loops throughout the test with little strength or stiffness degradation (Figure 6).

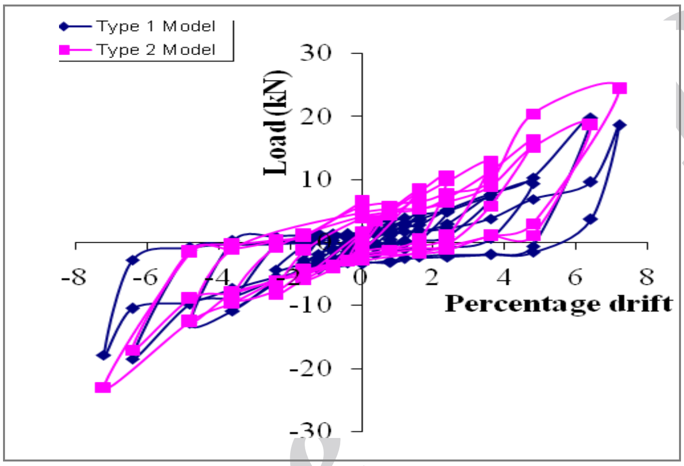

Figure 6. Load versus drift curve of specimens

6.2 Energy dissipation

Cumulative energy absorbed during each cycle of loading is plotted against corresponding cycle for Type 1 and Type 2 models as shown in Figure 7. It is observed that Type 2 model exhibits higher energy dissipation capacity compared to Type 1 model. The reinforcement passing through the joint had less robust anchorage for Type 1 model. In the case of Type 2 model, the anchorage of U bars passing through the connection was excellent.

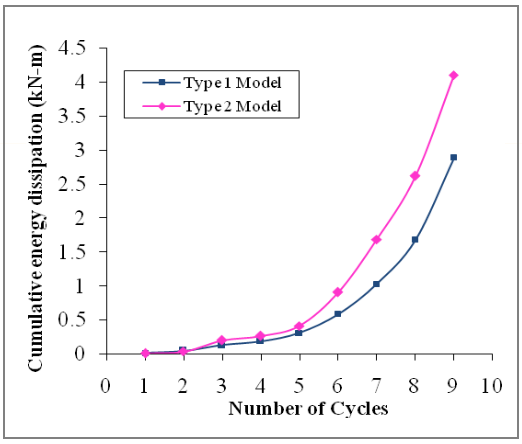

Figure 7. Cumulative energy dissipation curves of specimens

6.3 Displacement and Stress Contours

The maximum deformation was observed for the top surface of shear wall and it is gradually reduced towards the bottom. The deformed shapes under lateral loading are shown in Figure 8 and Figure 9. The Von Mises stress contours correspond to the two types of models (Type 1 and Type 2) are shown in Figure 10 and Figure 11. It was observed that the maximum stress occured at the bottom of the shear wall for both the types of models.

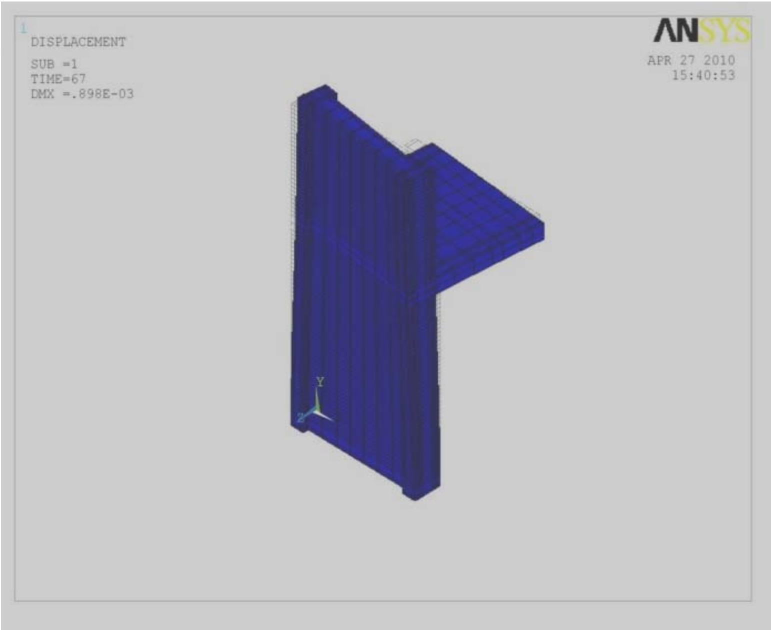

Figure 8. Deformed shape (Type1 Model)

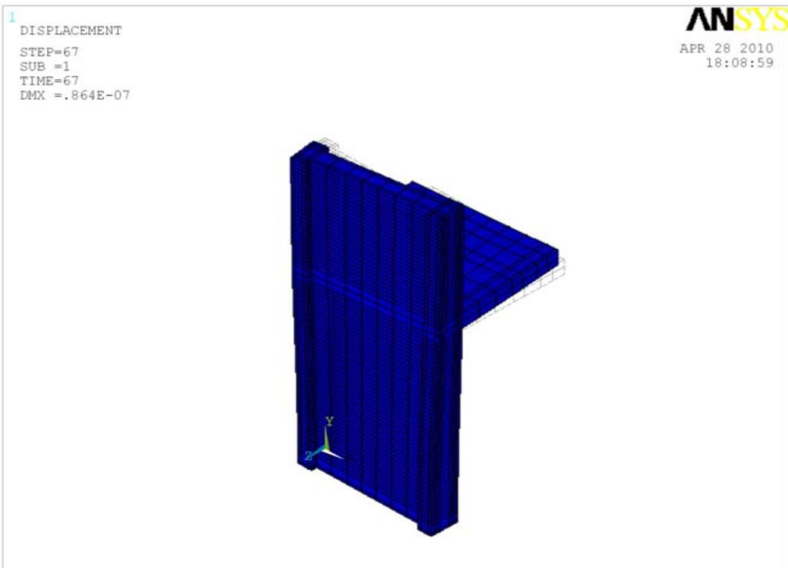

Figure 9. Deformed shape (Type 2 Model)

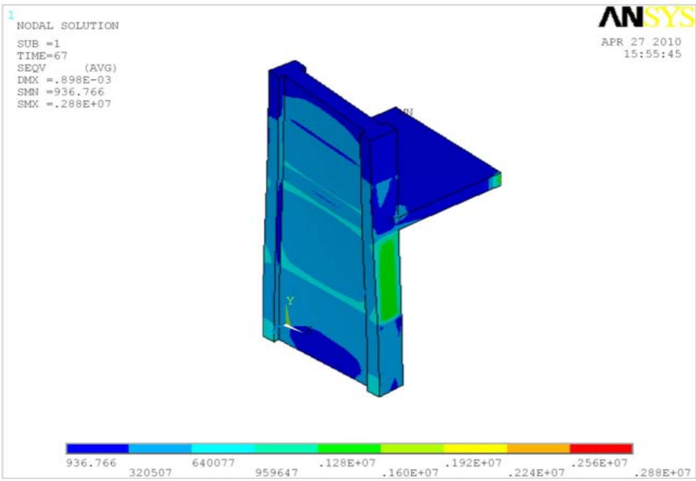

Figure 10. Von Mises stress for Type 1 Model

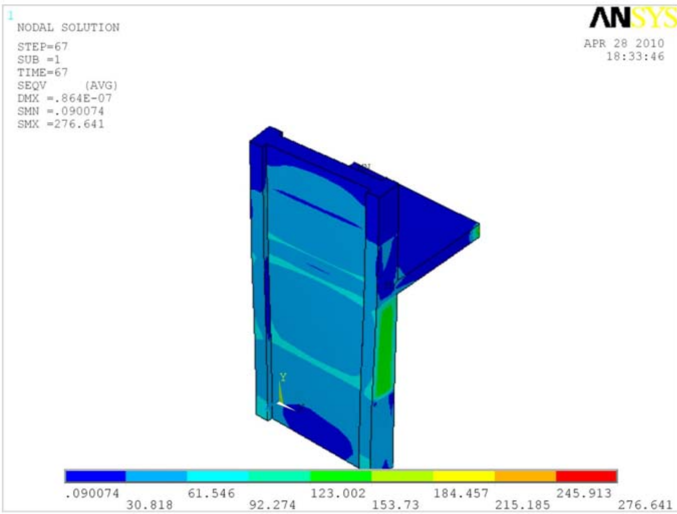

Figure 11. Von Mises stress for Type 2 Model

7. CONCLUSIONS

In this paper the effect of different parameters like ratio of height of shear wall to the effective width of the slab (H/W_e) on the behavior of exterior shear wall – slab joints was studied analytically. The shear wall and the slab has been modelled using ANSYS software, using Solid65 element and Link8 element.

Following are the conclusions drawn based on the analysis.

1. With respect to ultimate load, the specimen has exhibited higher ultimate strength when the confining U hooks are extended for an effective width of H/2.25 (Type 2) when compared with H/2.8 (Type 1). It is observed that the ultimate strength for Type 2 model is

12.3 % higher than that of Type 1 model.

2. Spindle-shaped hysteretic loops are observed with large energy dissipation capacity for Type 2 model compared to Type 1 model. The enhancement in energy dissipation for Type 2 model is observed to be 21.68 % higher than that of Type 1 model.

3. The maximum deformation was observed for the top surface of shear wall and it is gradually reduced towards the bottom. It was observed that the maximum stress occured at the bottom of the shear wall for both the types of models.

4. From all these observations, it is concluded that Type 2 model exhibited very good performance with respect to ultimate load capacity and energy dissipation capacity when compared to Type 1.

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