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Sverker Andreasson · Motoi Yasumura
Laurent Daudeville

# Sensitivity study of the finite element model for wood-framed shear walls

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**Abstract** A sensitivity study was performed with a nonlinear elastic finite element model for monotonic analyses of wood-framed shear walls. The objective was to provide information about simplifying a model of wood-framed shear walls with no significant loss in accuracy. The simplifications concern features such as slips in joints between frame members, slips in hold-down connections, and bearing between adjacent sheathing panels. The results from analyses of a shear wall with an opening of window shape show that the effect of constraint by the bearing between sheathing panels and slips in frame joints on the overall stiffness of the wall is limited. Thus, there are great possibilities for reducing the calculation time by not taking these phenomena into account, avoiding an excessive number of degrees of freedom and iterations. The influence of the simplifications on the distribution of vertical reaction forces along the wall is more significant. Furthermore, if each simplification is introduced separately, the effect on the overall stiffness is greater. The difference, however, is less than 10%. The failing pattern of the nail connections is also clearly influenced by the simplifications when they are introduced separately. The results from the analyses show that slips in frame joints can be sufficiently represented by those in connection with the opening.

S. Andreasson
Division of Structural Engineering, Lund University, Lund, Sweden

M. Yasumura (✉)
Faculty of Agriculture, Shizuoka University, Shizuoka 422-8529, Japan
Tel. +81-54-238-4863; Fax +81-54-237-3028
e-mail: afmyasu@agr.shizuoka.ac.jp

L. Daudeville
Laboratoire Sols, Solides, Structures, l'Université de Grenoble, Grenoble, France

**Key words** Wood-framed shear walls · Finite element model · Sensitivity study · Opening

## Introduction

There is growing interest in the development of commercial software in the field of wall diaphragm modeling. The main objective when developing calculation models for wood-framed shear walls is to achieve good accuracy with as little computational effort as possible. All models contain certain simplifications. However, it is essential to be aware of the effect of the simplifications on the calculation. Among the recent studies on modeling of wood-framed shear walls,[1–5] the effects of various simplifications in their models might have been studied, but few quantitative comparisons have been reported. It would be of great value for future model development to publish such knowledge on simplification effects.

In this study, a nonlinear elastic model presented by Davenne et al.[6] was applied in the analyses. A basic version of the model, consisting of pin-jointed frames and nonlinear nail joints, was compared with a modified version that included new features such as slips in frame joints and constraint by the bearing between adjacent sheathing panels. The basic model has the benefits of being plain, fast, and relatively accurate. The modified model is more complex and thus requires much more calculation time. For the analyses, each feature was separately introduced and evaluated to estimate the effect of the respective modification on the results.

Based on the findings in this study it was possible to estimate which features are advisable for a shear wall model and which features can be justifiably neglected. It is assumed that the effect of features such as slips in frame joints and bearing between sheathing panels is of more significance for structures with openings than for continuous wall elements. Consequently, a plywood-sheathed shear wall with a window opening was used as the reference object for the finite element analyses. The results are also examined in relation to experimental values to confirm the validity of the model.

## Modeling

### Basic model

A shear wall was modeled with beam, plate, and spring elements representing frame members, sheathing panels, and fasteners connecting the sheathing panels to the frame, respectively. The frame elements were modeled with isotropic linear two-dimensional (2D) beam elements. The sheathing panels were modeled with 2D plane stress elements that were elastic and orthotropic. The fasteners between sheathing panels and frame beams were modeled with a nonlinear spring system.[6] The force–deformation curves of fasteners followed a trilinear curve. The nodal forces were related to the relative nodal displacements. In the basic configuration, joints connecting frame members were modeled as pin joints. When hold-downs were left out at the opening, it was modeled by releasing the coupling between the studs and bottom plate at the joints at a tension closest to the opening. Finite element code CASTEM 2000 was applied to the modeling and calculation.

### Features of the modified model

To represent the real behavior of a shear wall more accurately, a number of features were introduced to the basic model. First, the pin joints connecting frame members were replaced with spring elements: two linear springs in the vertical and horizontal directions and one linear spring in the rotational direction. The springs were independent each other in terms of compression, tension, shear, and rotation. Initial gaps in the frame joints were not considered in the model. The flexibility of hold-down connections was modeled by introducing a tension stiffness of the particular frame joints corresponding to the hold-down stiffness obtained from small specimen tests. At the locations of the hold-down connections, the bottom plate was assumed to be rigidly connected to the foundation. In the analyses neglecting slips in hold-down connections, the hold-down stiffness was set close to infinity. Constraint by the bearing between adjacent sheathing panels was modeled by unilateral constraint equations between the horizontal displacements of coincident panel corners. In the case of "compression" the horizontal displacements at the corners were equal. In case of "tension" there was no such constraint. These conditions were determined within each iteration at each load increment. Bearing was assigned only to the corners of the panels to minimize the extra computational time needed for this feature. At the small panels above and underneath openings, the constraints were set between these panel corners and the coincidental nodes, at the adjacent panel edge, which were not necessarily corner nodes, as shown in Fig. 1A. Because of the choice of restraining only the horizontal degrees of freedom at the panel corners, there is a slight error (Dx) in the contact definition, as the corner points also move in relation to each other in the vertical direction, as shown in Fig. 1B. This error, however, is regarded as insignificant for the load case and wall configuration investigated in this study.

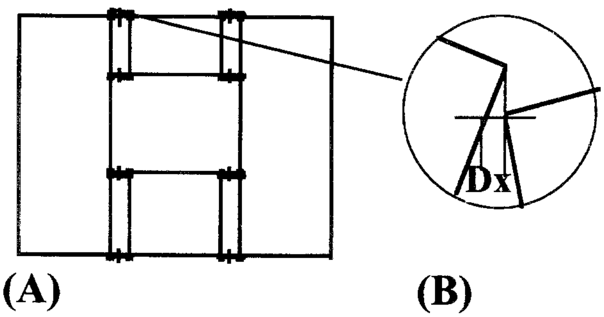



**Fig. 1.** Model representing constraint by the bearing between adjacent sheathing panels. **A** Location of constrained nodal points. **B** Error ($Dx$) due to the simplified bearing used

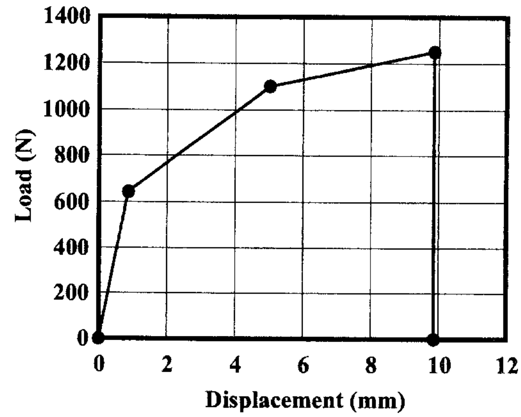



**Fig. 2.** Trilinear approximation of nail properties used for finite element analyses

### Material properties

Poisson's ratio and the modulus of elasticity (MOE) of the frame members were assumed to be 0.2 and 10.0 GPa, respectively. For the panel elements, Poisson's ratio, MOE, and the shear modulus were assumed to be 0.5, 5.3, and 0.7 GPa, respectively. The nail properties used for the fastener elements was determined by nail joint tests, as performed by Yasumura and Kawai.[7,8] Trilinear approximation, as shown in Fig. 2, was adopted for the analysis. The embedding stiffness of the frame joints and the tension stiffness of the hold-downs were determined by tests performed with static compression loading, as shown in Fig. 3. The embedding and hold-down tests were performed with three specimens for each configuration.

In the hold-down test performed with symmetrical specimens, HDB-20 hold-downs with four lag screws 12 mm in diameter and 75 mm long were used. Two hold-down connections were placed on a stud package of four 38 × 89 mm studs. Assuming that the displacement is primarily assigned

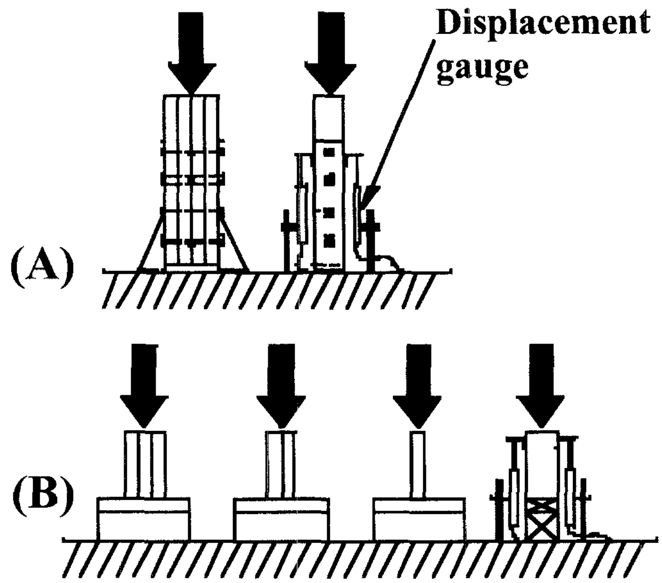



Fig. 3. Configuration of tests. A Hold-down test. B Embedding tests

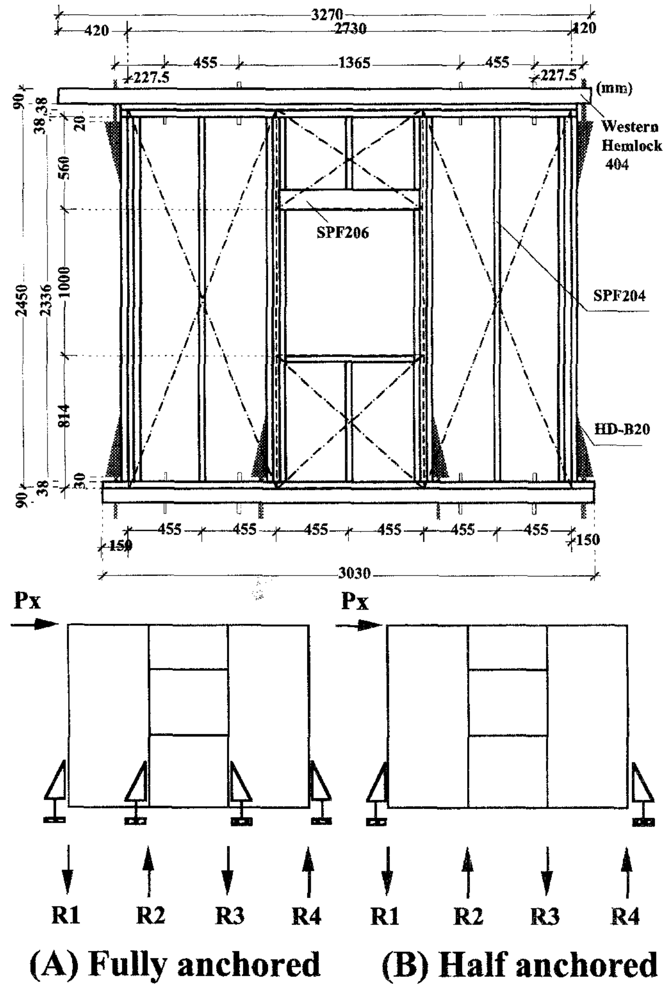



Fig. 4. Configuration of the wall specimens studied. A Fully anchored shear walls. B Half-anchored shear walls

to the slip of the joints, the joints were tested in compression instead of tension, as shown in Fig. 3A. The embedding tests were performed with three different configurations made up of one, two, or three studs, as shown in Fig. 3B. The results from the tests were evaluated to find a suitable linear approximation to be implemented in the finite element model. The approximations of embedding stiffness were 12.8, 21.2, and 23.7 MN/m for one, two, and three studs, respectively. The stiffness for the three-stud specimen was chosen for the model, as the largest forces arises where three studs are mounted together. The hold-down stiffness was determined to be 13.1 MN/m.

Preliminary results showed that the effect of lateral slips on the frame joints was moderate. The lateral stiffness of the joints was thus modeled to be rigid in this study. Furthermore, the rotational stiffness in the frame was disregarded in the analyses. It is assumed that this stiffness is small compared to the stiffness provided by the panels being attached to the frame. The rotational stiffness of the hold-downs might be more significant. However, it is difficult to estimate this stiffness because it depends on the loading direction and the deformation of the framing. Consequently, it was decided not to take this feature into account.

## Analyses

To evaluate the influence of the various features of the model, a plywood-sheathed shear wall of a size corresponding to three panels with a window opening in the middle panel, as shown in Fig. 4, was used as the reference object. In the following finite element analyses, each modification was added separately to the basic pin-jointed model to investigate the effect on the results. The model with all new features was analyzed and compared to the basic model. The analyses were performed with a conventionally anchored wall and a half-anchored wall. The monitored parameters were the overall load-displacement characteristics, the distribution of vertical reaction forces, and the order in which the nails reached their maximum capacity.

### Configuration of analyzed shear wall

The analyzed walls were wood-framed shear walls sheathed with 9.5 mm thick C-S-P plywood built up in three layers. The full size panels were 910 × 2450 mm. The plywood was attached to the framing with CN50 nails, with 100 mm spacing at the perimeter and 200 mm at intermediate studs. The studs, top and bottom plates, and bottom rail at the window consisted of 38 × 89 mm members of S-P-F. The lintels were made from a 38 × 138 mm member. The double top plates were attached to a 38 × 76 mm ledger with four bolts 16 mm in diameter and the bottom plate to the steel frame through a 38 × 76 mm sill of western hemlock. At the wall ends three studs were put together to form a stud package. At the opening the stud packages consisted of two studs above the window opening.

In the wall configuration with hold-downs at both wall ends and opening ends, four hold-downs of HDB-20 were attached with four lag screws 12mm in diameter and 75mm long. In the wall without interior hold-downs, two HDB-20 connectors were used at both wall ends. The bottom plate was attached to the sill and the top plate to the ledger by intermediate bolts in the two outermost joist spacings. In the finite element model the bottom plate was assumed to be pin-jointed at the location of each hold-down and at the positions of the intermediate bolts between the bottom plate and the sill plate. In the model, the position of the latter was adjusted to the closest nodal point to avoid the need of overly dense mesh; this was considered a minor adjustment. The top plate and the ledger were assumed to interact although not completely. The flexural rigidity was assumed to be equal to the sum rigidity of the double top plates and the ledger.

### Sensitivity study

In the basic model, all joints connecting frame members were considered to be pin joints. Therefore, the slips in the framing joints were considered during the first analyses. The slip in the tension of the joints was introduced in several steps. For the first step the tension stiffness was set to zero only at the joints at the window opening, as shown in Fig. 5A. Second, zero stiffness was introduced only in the joints at the top and bottom plates between studs and the top and bottom rails, as shown in Fig. 5B. In the third case, as shown in Fig. 5C, zero stiffness was introduced in all framing joints simultaneously. Some analyses were also performed with a tension stiffness corresponding to the initial stiffness seen when pulling out nails. It is assumed, however, that the zero stiffness approach is more accurate because the pulling-out stiffness quickly decreases at moderate displacement.

In the following analyses, the framing joints were modeled with a compression stiffness corresponding to the embedding stiffness of cross-grain compression derived from small specimen experiments accounted for in the previous section. In the model, the same stiffness was introduced at all joints even though the number of studs differed at different joints. The stiffness was derived from the experiments with three-stud packages. This stiffness is also representative of stud packages of two studs. It is approximately twice the stiffness for embedding when using only one stud. It is assumed that this is of minor importance, as the largest compression forces occur in the bottom plate at wall ends and openings.

Next, nonrigid hold-downs were evaluated. When introducing flexibility in tension for the hold-down connections, it is, of necessity, combined with flexible framing joints. Constraint by the bearing between adjacent sheathing panels is modeled with pin-jointed framing joints, as in the basic model. In the final analyses all features (i.e., bearing, flexible joints, embedding, and flexible hold-downs) were introduced at the same time.

## Results and discussion

### Overall wall stiffness

The effect of each introduced feature on the load-displacement characteristics for the fully anchored wall is plotted in Fig. 6A. It can be seen that the largest consistent effect is that of the bearing between adjacent sheathing panels. The bearing gives an increase in wall stiffness of about 6%. The other modifications that produce different flexibilities in the framing joints reduce the stiffness somewhat during the initial phase, by up to 12%. A comparison of the flexibility in tension at all joints, at only the window rail joints and at the top and bottom plates showed that the effect of flexibility is almost exclusively seen in the joints at the window rails. If all new features are introduced it can be seen that the deviation between the basic model and the modified model is small at larger displacements. The deviation is more pronounced at the beginning of the loading.

The deviation is somewhat larger for the wall configuration without hold-downs at the window opening. The difference in deviation, however, is less than 3% between the two wall configurations. In Fig. 6B it can be seen that introduction of a finite embedding stiffness or hold-down stiffness seems to increase the overall stiffness. Furthermore, the overall stiffness increases when flexibility in the frame is introduced. This is due to the fact that the frame joints closest to the opening are released in the basic model, whereas they are modeled bilinearly in the modified model. The sudden decrease in capacity that can be seen in the analyses corresponding to flexible joints in regard to all options is due to the fastener formulation (i.e., the negative slope after the maximum load is not included). The capacity of a fastener drops immediately from maxi-

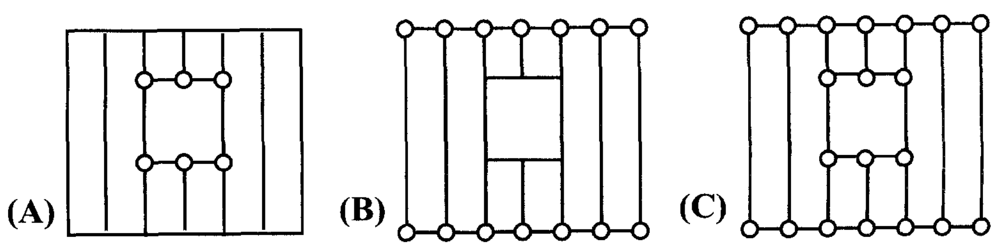



**Fig. 5.** Location of frame joints where flexibility is introduced. **A** At opening. **B** At top and bottom plates. **C** At all joints

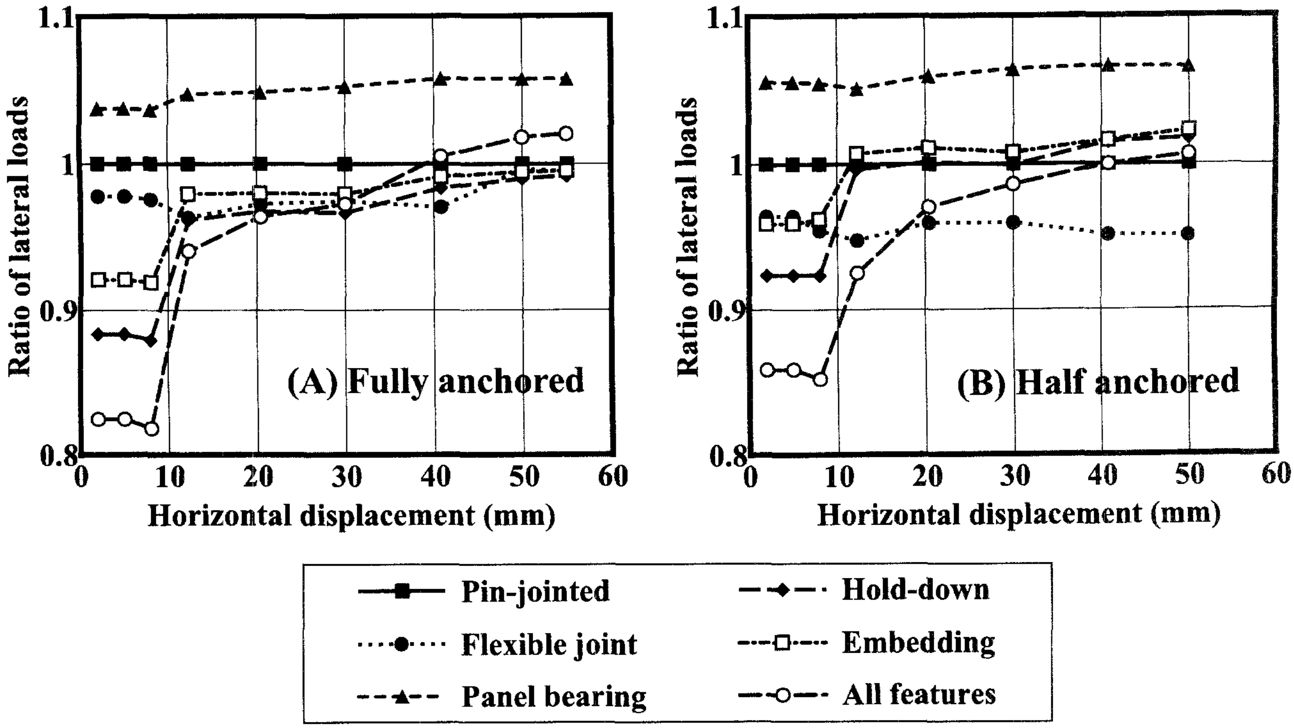
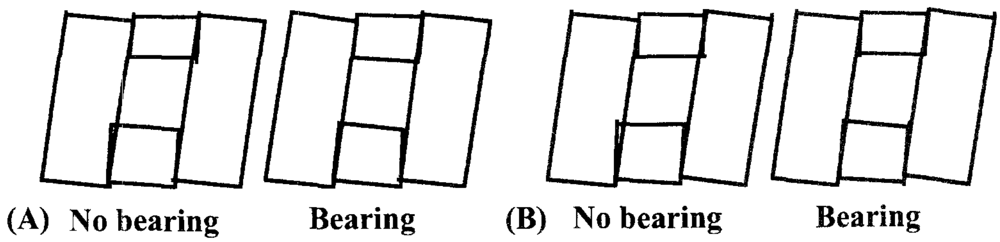



**Fig. 6.** Effect of each modification as well as all of them altogether for fully anchored (**A**) and half-anchored (**B**) shear wall

**Fig. 7.** Panel displacement in deformed meshes without and with bearing. **A** Fully anchored shear wall **B** Half-anchored shear wall. (The displacements are magnified 10 times)




mum to zero. This point is reached at an earlier stage in a wall without interior hold-downs than in a fully anchored wall.

As mentioned previously, the bearing effects increased the wall stiffness. When bearing was introduced, the rotation of the panels above and below the window opening increased up to three times for the panel above the opening. The full-size panels were not significantly influenced, as shown in Fig. 7. If the results are considered in relation to the experimental values obtained from a previous study by Nagata and Yasumura,[9] it can be seen that both models are relatively accurate, as shown in Fig. 8. The difference in overall stiffness between the basic model and the modified model, with all new features introduced in the model, is insignificant. The only major difference concerns the maximum load. In the model with flexible joints and bearing, the nails reach their maximum capacity in a slightly different order, with earlier failure at a lower load. This is more pronounced in the wall without interior hold-downs than in the fully anchored wall.

Vertical reaction forces

It is interesting to note that the hold-down forces and the compression forces deviate significantly from the conventional approximations used in practical design. This is in accordance with previous studies (e.g., Andreasson[10]). Commonly, the small walls above and below the opening are generally disregarded, and the wall parts with full-size panels are simply supported, resulting in vertical reaction forces of the same magnitude at all panel ends. In the analyses, however, it is evident that the force distribution in a wall with a window opening is quite different, as shown in Table 1. In particular, the reaction forces at the opening are significantly smaller than the predictions based on conventional design.

When introducing the new model features individually as well as together, the vertical reaction forces are redistributed as shown in Table 2. For a fully anchored wall, the forces at the opening (R2, R3) are most influenced. If flexibility in tension is introduced, these forces increase by about 15%. The largest redistribution concerning reaction forces is obtained when introducing bearing between sheathing panels. It produces a reduction of about 33% for the fully anchored wall. When all the new features are introduced, the largest deviation is a 30% reduction at the compression side of the opening. The redistribution is somewhat more complex for the half-anchored wall. When applying all modifications, the largest difference is an 18% reduction of the compression force at the opening.

**Table 1.** Relative vertical reaction forces at the bottom plate supports obtained in analyses with the basic finite element model

| Configuration | Reaction force (%)[a] | | | |
|---|---|---|---|---|
| | R1 | R2 | R3 | R4 |
| Fully anchored | 77 | 32 | 32 | 77 |
| Half-anchored | 78 | 19 | – | 67 |

The forces are given in percent of calculated values relative to those in current design principles neglecting the effect of opening
[a]For explanation of R1, R2, R3, and R4 see Fig. 4

It is evident that the distribution of vertical reaction forces can be influenced by modifications of the model despite the fact that the overall load-displacement stiffness is unaffected. When comparing the results for stiffness and force distribution at a storey drift of $^1/_{60}$ when all features are introduced, the overall stiffness is approximately equal for the basic and modified models, whereas the force distribution at the same time by as much as differs 30%. Moreover, the reaction forces deviate significantly from the ones stated by current design methods.

*Nail stress pattern*

To investigate any changes in nail stress caused by the features introduced into the model, the nail failing order was monitored. The order in which the nails reached their maxi-

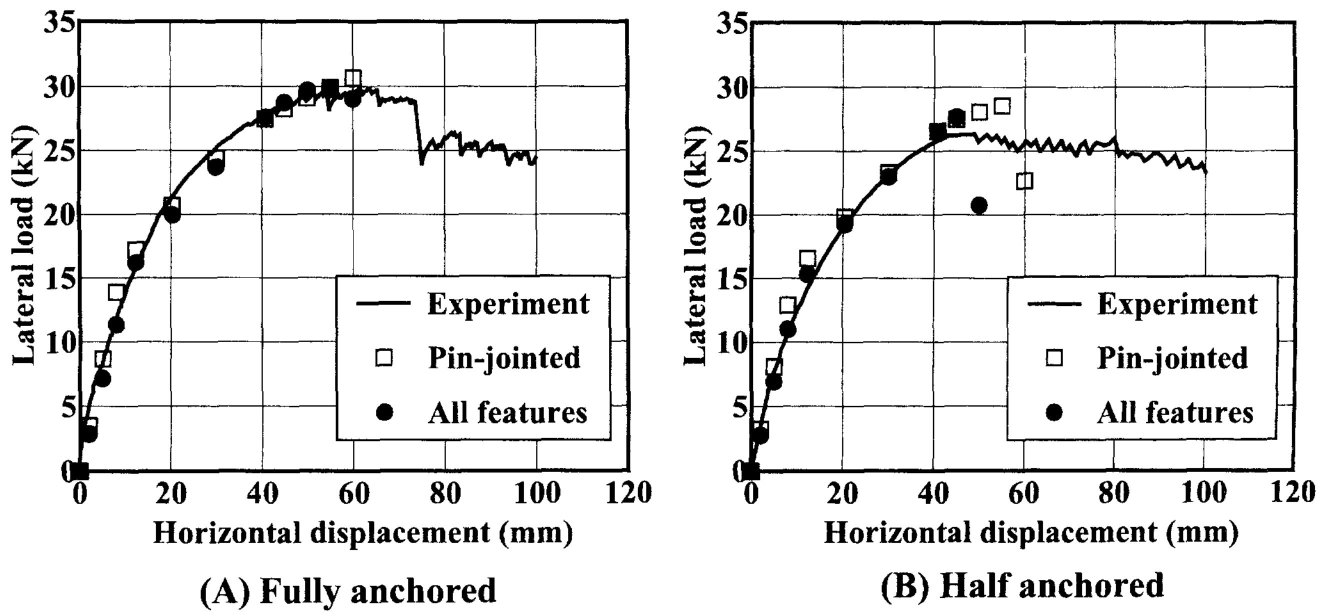



**Fig. 8.** Comparison of experimental results with calculation by basic and modified finite element models for fully anchored (**A**) and half-anchored (**B**) shear wall

**Table 2.** Influences of model modifications on the distribution of reaction forces in fully anchored and half-anchored shear walls

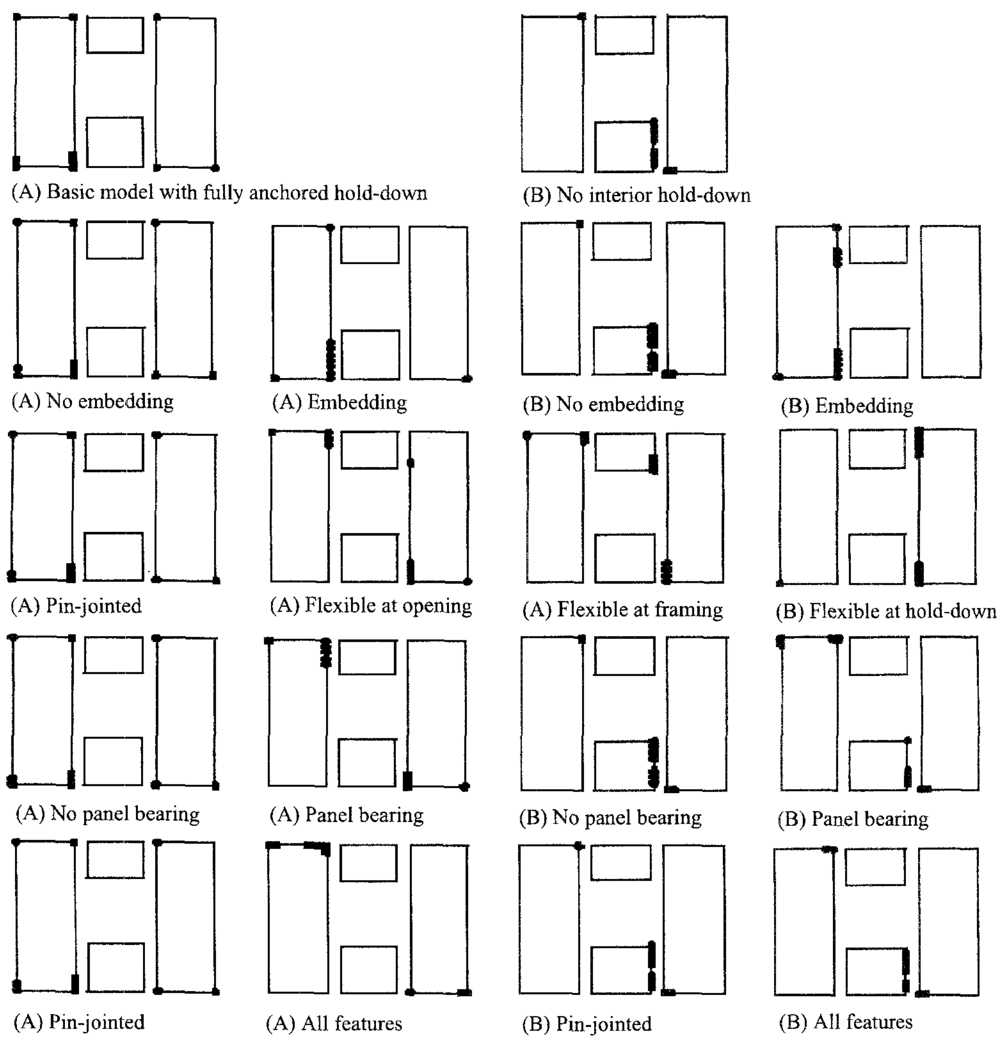
| Feature | Fully anchored[a] (kN) | | | | | Half-anchored (kN) | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | Px | R1 (%) | R2 (%) | R3 (%) | R4 (%) | Px | R1 (%) | R2 (%) | R3 (%) | R4 (%) |
| Pin jointed | 27.46 | 28.43 (77) | −11.73 (32) | 11.69 (32) | −28.42 (77) | 26.52 | 27.90 (78) | −6.85 (19) | – | −24.06 (67) |
| Flexible joints | 26.65 | 28.20 (79) | −12.88 (36) | 13.0 (36) | −28.03 (78) | 25.23 | 27.64 (81) | −8.67 (26) | – | −22.75 (67) |
| Bearing | 29.06 | 28.53 (73) | −8.27 (21) | 8.41 (21) | −28.76 (74) | 28.29 | 27.93 (73) | −4.52 (12) | – | −25.21 (66) |
| Flexible hold-downs | 27.02 | 25.44 (70) | −10.74 (30) | 9.29 (26) | −28.68 (79) | 26.93 | 25.34 (70) | −7.76 (21) | – | −29.29 (81) |
| Embedding | 27.23 | 28.58 (78) | −10.17 (28) | 11.42 (31) | −26.85 (73) | 26.95 | 28.41 (78) | −6.83 (19) | – | −27.08 (75) |
| All options | 27.61 | 26.19 (70) | −8.31 (22) | 8.61 (23) | −27.18 (73) | 26.54 | 26.02 (73) | −5.62 (16) | – | −23.33 (65) |

The results are given for an apparent shear deformation of $^1/_{60}$
Values in parentheses are the vertical forces/expected values in current design procedure
[a]For an explanation of Px, R1, R2, R3, and R4 see Fig. 4

mum load capacity was surveyed. The first ten nails to reach maximum load were evaluated and comparisons were made between analyses. There was a general trend that the number of nails reaching the maximum load at the same load increment was much larger for the walls with a conventional hold-down configuration than for the walls without interior hold-downs. It is thus concluded that the nail forces are more equally distributed in the fully anchored wall. This seems reasonable, as the force distribution ought to be more complex in the perforated wall because of the asymmetrical stiffness at the up-lifting studs.

Our analyses showed that the nail-failing pattern is significantly changed when new features are introduced in the model. First, it was noted that the exposure of the nails differs significantly between a wall with hold-downs at both the wall ends and the opening and a wall with no hold-downs at the window opening. In the latter case, the main failure takes place in the panel under the window, whereas the most exposed nails are found at the full-size panels if hold-downs are fully anchored, as shown in Fig. 9. This is due to the fact that the studs at the opening are free to lift in the second case, which increases the deformation of the fasteners at the up-lifting stud. When flexibility in compression is introduced to take embedding of the frame into account, it can be seen that this gives a more exposed situation for the nails at the first compression stud at the left side of the window opening. When flexibility in tension is introduced in the frame, in both the frame joints and hold-downs, a more severe situation for the nails is created at the up-lifting point of the window opening. This phenomenon also occurs when only the joints at the window rails have flexible tension. For the wall without hold-downs at the opening, no trend correlated with the introduction of the flexibility in tension. This is also true for the effect of the bearing between sheathing panels. A clear trend can be seen for the fully anchored walls, however. The bearing between the panels increases the stress of the nails at the outermost contact points. When all new features are introduced simultaneously in the model, the main observation is that the nail failing order agrees well for the simple and more advanced models. As mentioned previously, this is also true for the overall load-displacement relation of the wall. The probable explanation for this similarity, despite the many differences between the models, is that some of

**Fig. 9.** Nail-failing pattern in various models. **A** Fully anchored shear wall. **B** Half-anchored shear wall



the features counteract each other. For instance, the effect of the bearing makes the wall stiffer, whereas the effect of flexible frame joints makes the wall less stiff.

## Conclusions

Results from analyses of a discontinuous wall with a window opening show that the effect of constraint by the bearing between adjacent sheathing panels and slips in framing joints on the overall stiffness of the wall is limited (less than 10%). Among the various features investigated in this study, bearing between adjacent sheathing panels has a pronounced influence on the results. If bearing is introduced, the overall stiffness increases by approximately 6%. The vertical reaction forces decrease by approximately 30% at the interior supports. Because bearing increases the overall stiffness and decreases the reaction forces, it is safe to disregard the effect of bearing. Furthermore, it is often desirable to avoid bearing between panels in real construction to prevent buckling of sheathing panels due to moisture variation.

Flexibility in the tension of frame joints and hold-downs is the second most important feature. In this case the magnitude of the impact is about 3%–15% for wall stiffness and reaction forces. To take flexibility in frame joints into consideration, it is sufficient to introduce such flexibility at only the opening rails if the wall is fully anchored. Flexibility in the joints at the top and bottom plates at other positions causes no significant difference. The effect of the flexibility of frame joints, and to some extent the effect of bearing, is more pronounced in the wall without hold-downs at the opening.

When all features are introduced in the model, the difference in overall stiffness is 1% at most. This is due to the fact that some features counteract each other. However, the effect of the vertical reaction forces might still be significantly influenced by these features. In our analyses, the deviation was about 30% at the interior supports and 8% at the end supports.

If flexibility in joints and bearing between sheathing panels is considered, the nail failing pattern is somewhat more critical. The most exposed nails reach their maximum capacity at a lower load.

Finally, it was noted that the distribution of vertical reaction forces deviates significantly from that in conventional design approaches. The forces at interior and end supports might be as low as 20%–70% of the predicted ones. This is also true when using the basic pin-jointed model.

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