Discussion of "System Identification and Damage Detection of a Prestressed Concrete Beam" by Jörg F. Unger, Anne Teughels, and Guido De Roeck

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The discussers appreciate the authors' comprehensive work to develop a system identification method for prestressed concrete beams. The method consists of experimental modal analysis and a model updating technique and is successfully applied to detect damage in a prestressed concrete beam. Although the developments are clear and the experimental data are reliable, the discussers would like to seek clarification on several issues.

This paper is in fact a companion to Unger et al. (2005). Some of the contents are similar to those of the previous paper, but neither paper provides static damage data such as crack distribution, deflected shape, and failure mode under static loading. The authors describe how ultimate failure occurred at two sections 2 m apart (Unger et al. 2005), midspan of the beam, and this description is used to support the results of model updating indicated in Figs. 8(e)–(f) in the discussed paper, but relevant experimental evidence is not shown in the paper. The only picture (Fig. 3) in the paper seems to illustrate distributed damage rather than concentrated plastic hinges because the distance between the two cracks in Fig. 3 appears to be much smaller than 2 m.

In Fig. 6 in the paper, the mode shapes calculated with the FE model ($\text{Beam}_{\text{free}}$) are compared to the measured mode shapes. Discrepancies can be seen in the three curves, but the model updating the measured mode shapes include average curves of two measurement lines (front and back). If the average curves were plotted in Fig. 6 in the paper, it is believed that the discrepancy in the two curves (calculated and the measured average curve) is very small.

The authors consider that the weighting of the different residuals (mode frequencies and shapes) has a strong influence on the results of model updating, so different weight factors are used for mode frequencies and shapes based on the measurement error. Moreover, the value of $W_{m/f}$ is taken as 4, which means the mode frequencies have a larger influence on the results of model updating than the mode shapes. But from Table 1 and Fig. 6 in the paper, the difference between calculated and measured frequencies appears to be larger than that of the mode shapes. The paper points out that the MAC values defined in Eq. (6) of the paper for the first three mode shapes are nearly 100%. The discussers would like to question how much these values are before model updating if the measured average mode shapes are used. Furthermore, in the companion paper (Unger et al. 2005) the ratio of weight factors was taken as 5, but the results of model updating are almost the same, which means that the parameter identification is insensitive to the weight factors, at least in a range of 4 to 5.

The Young's modulus of the beam, obtained from material testing, is, $3{,}350\ \text{kN/cm}^2$, and it is noticed that the material testing method is a kind of dynamic method. The value of Young's modulus should be used in FE for calculating mode frequencies listed in Table 1 in the paper, and from Fig. 9(a) in the paper it can be seen that for the undamaged beam, the updated Young's modulus of almost all the elements is smaller than the result of material testing. Concrete shrinkage, inhomogeneous properties, and so on might be the possible reasons, but there is no convincing evidence to explain the difference between material testing and prestressed concrete beam testing. If the value of Young's modulus is raised by 15%, the calculated frequencies could be within a range of 1% of the measured frequencies for the model $\text{Beam}_{\text{free}}$. For the undamaged beam, the results of model updating only reflect the material properties; can we therefore find a better way to determine the dynamic modulus of concrete?

Many factors are not considered in the analysis, such as shear deformation, reinforcement, prestressing effect, model error, and so on. Using the data listed in Table 1, the relative difference between calculated and measured frequencies can be plotted as the curves shown in Fig. 1, which shows that different models cause different errors. Note that the difference increases with the order of mode, which may be due to shear deformation or other reasons for which the paper provides no explanation. It is also interesting that in Fig. 9 in the paper, the Young's modulus of the elements near the two ends of the beam changes for different load steps. It is generally well known that the flexural stiffness does not change with applied load in the end regions of a prestressed concrete beam, so the minimization of the objective function by model updating only has mathematical significance, and thus the structural behavior should be further emphasized.

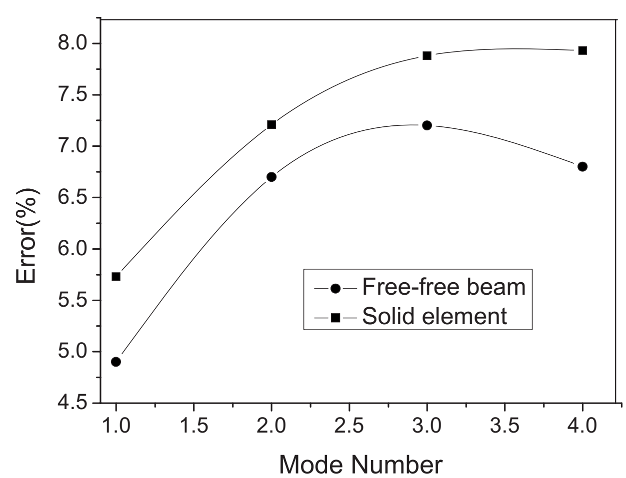

Fig. 1. Relative difference between calculated and measured frequencies with different mode numbers

References

Unger, J. F., Teughels, A., and De Roeck, G. (2005). "Damage detection of a prestressed concrete beam using modal strains." *J. Struct. Eng.*, 131(9), 1456–1463.

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It is quite common in the updating process to give more weight to the eigenfrequencies than to the mode shapes. Fig. 5 in the discussed paper shows that damage will produce small but nevertheless clear changes in the mode shapes, so they should be included in the objective function with an appropriate weight. When applying a relative weight factor of 4, excellent agreement was obtained for the eigenfrequencies as well as for the mode shapes. The correction factors resulting from the updating for the initial state and for the different load steps are given in the discussed paper in Fig. 9, and the corresponding bending stiffness distribution in Fig. 8.

However, because of an Erroneous interpretation of Figs. 8 and 9 and by overlooking Table 1, the discussers mistakenly conclude that for the initial state, a lower Young's modulus is obtained in the beam experiment than was derived from a dynamic test on a cylindrical sample. The reverse is true. The reason for a higher Young's modulus can be explained by the effect of prestressing in the test beam (not present on the cylindrical sample) and the fact that the passive reinforcement has not been explicitly modeled (its influence will be rather small, taking into account the amount of reinforcement). The stiffness-increasing effect of prestressing is due to the closing of some small microcracks.

The authors do not agree that shear deformation, reinforcement, and prestressing effects have not been considered in the analysis. A solid model with volume elements produced comparable eigenfrequency values in the considered frequency interval. It is obvious that this solid model includes shear deformation. Moreover, shear deformation was considered in the beam models as well. The reason to apply updating to a beam model and not to a solid model is the ability to work with a reduced set of physically meaningful updating parameters and, on the other hand, to use the type of numerical model that will also be used in engineering practice for this kind of prestressed concrete beams. The (small) amount of passive reinforcement resulted in a slight increase in the effective Young's modulus. The geometric stiffness effect has been investigated but proved to be negligible, as already mentioned in the discussed paper.

Discussion of "Framed Steel Plate Wall Behavior under Cyclic Lateral Loading" by Hong-Gun Park, Jae-Hyuk Kwack, Sang-Woo Jeon, Won-Ki Kim, and In-Rak Choi

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The authors conducted interesting tests on steel plate shear walls. Shear and flexure are simplified in such a way that, when used for design purposes, may lead to rough estimations. The displacement of thin or thick steel plate shear walls in an elastic region is a combination of shear and flexure displacements at any level (Fig. 1).

If flexure is not rationally considered in the design, the system would fracture before forming any plastic shear deformation, even in thin steel plate shear walls. On the other hand, as shown in Fig. 2, the hysteresis loops of a framed steel plate shear panel are a combination of hysteretic curves of plate and frame (Roberts and Sabouri-Ghomi 1991; Sabouri-Ghomi and Roberts 1992). So any introduced analytical model must have the ability of predicting these curves.

The developed plastic moments of the frame members shown in Fig. 2(c) and consequently Eq. (2) in the discussed paper are not always correct. The locations and sequence of forming plastic moments depend on many parameters of the steel plate shear walls (Sabouri-Ghomi and Gholhaki 2006).

Eq. (3) in the discussed may be used only for very thin plates. When the plate is considerably thick, the critical shear stress (τ_{cr}) must be considered in that equation (Sabouri-Ghomi et al. 2005; Sabouri-Ghomi 2002; and Roberts and Sabouri-Ghomi 1991).

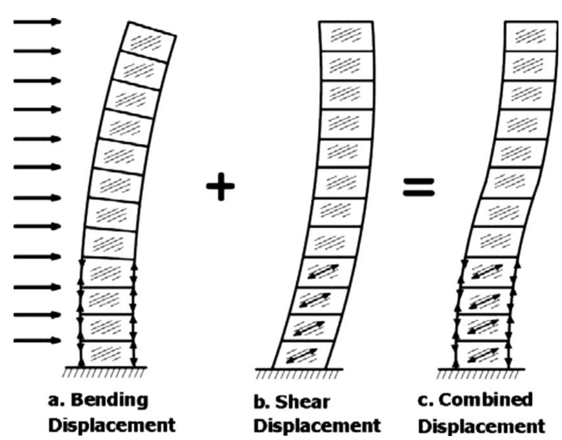

Fig. 1. Bending and shear combination in SPSW

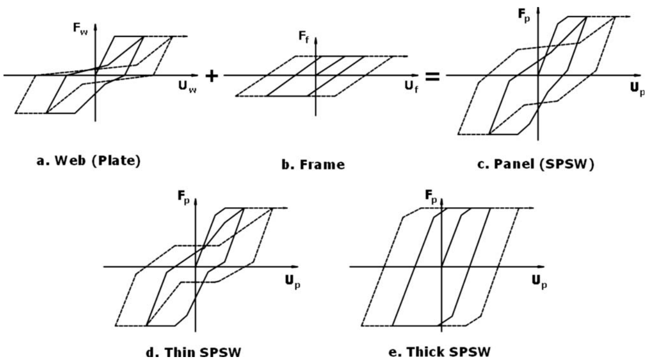

Fig. 2. Load-displacement hysteresis model of plate, frame, and panel (SPSW)

The buckling shear strength has a significant role in the hysteretic behavior of steel plate shear walls (Fig. 2). Furthermore, as shown in Figs. 2(d and e), when cyclic loading is of concern for nonlinear dynamic analysis, τ_{cr} cannot be ignored at all. On the other hand, when the plate thickness is significant, the flexure stresses in the steel plate are considerable and cannot be ignored in the calculations. So Eqs. (3) and (4) in the discussed paper must be modified.

Based on the above discussion, the V_s/V_f ratio for recognizing the flexure or shear mode of the steel plate shear walls is rough and hence not reliable.

The respected authors of the paper concluded that "the failure of well-designed steel plate walls occurred at the column base or at the beam-column connections." We are of the opinion that this conclusion is dangerous, since any damage to the main members of a structure may lead to its collapse. So we believe that in conclusion it should be stated that "well-designed steel plate shear wall is that in which most of energy is absorbed by the steel plate until it fractures and where the other members, such as columns, are protected from any damage."

If the plate-frame interaction was considered precisely in the design of tested steel plate shear walls, even with thick steel plates, the columns could be protected from any damage. In this regard, the employment of the general concept of easygoing steel (EGS) could be beneficial (Sabouri-Ghomi et al. 2005; Sabouri-Ghomi et al. 2004, 2002).

The plate-frame interaction technique was used to examine the tested specimens. It was seen that in all the specimens the frames became plastic in advance of the plates. So, as expected, the frames experienced large plastic deformations even in walls with thin plates.

Finally, it is suggested that the respected authors present the first and second load-displacement curves of the tested shear walls. These curves may be used for better judgment of the results.

References

Roberts, T. M., and Sabouri-Ghomi, S. (1991). "Hysteretic characteristics of unstiffened plate shear panels." *Thin-Walled Struct.*, 12, 145–162.

Sabouri-Ghomi, S. (2002). *Lateral load resisting systems: An introduction to steel shear walls*, Anguizeh Publishing Ltd., Tehran, Iran.

Sabouri-Ghomi, S. (2004). *Lateral load resisting systems: An innovative idea to application of easy-going steel*, Anguizeh Publishing Ltd., Tehran, Iran.

Sabouri-Ghomi, S., and Gholhaki, M. (2006). "Cyclic tests on two specimens of three-story ductile steel plate shear wall." *Rep. Sent to Building and Housing Research Center (BHRC)*.

Sabouri-Ghomi, S., and Roberts, T. M. (1991). "Nonlinear dynamic analysis of thin steel plate shear walls." *Comput. Struct.*, 39(1/2), 121–127.

Sabouri-Ghomi, S., and Roberts, T. M. (1992). "Nonlinear dynamic analysis of steel plate shear walls including shear and bending deformations." *Eng. Struct.*, 14(5), 309–317.

Sabouri-Ghomi, S., Ventura, C. E., and Kharrazi, M. H. K. (2005). "Shear analysis and design of ductile steel plate shear walls." *J. Struct. Eng.*, 131(6), 878–889.

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The authors would like to thank the discussers for their interest in this paper. Each of the questions and comments presented by the discussers is discussed separately, as follows.

1. The discussers indicated that the developed plastic moments of the frame members shown in Fig. 12 and consequently Eq. (2) of the discussed paper are not always correct. The authors agree on the discussers' comment. However, as indicated in the paper, Eq. (2) and Fig. (12) do not show the plastic mechanism of frames arbitrarily designed, but rather the plastic mechanism of the specimens tested in the present study, for the purpose of predicting the load-carrying capacities of the specimens.
2. Furthermore, the discussers indicated that when the plate is considerably thick, the critical shear stress (τ_{cr}) must be considered in that equation. As clearly indicated in the abstract, introduction, and conclusions of the paper, this experimental study focused on the investigation of the cyclic behavior of the steel plate walls with thin infill plates. In the walls with thin infill plates, at very early loading, tension-field action is developed by shear buckling of the infill plates. Therefore, the shear buckling stress of the infill plates need not be considered in the calculation of the ultimate load-carrying capacity and deformation capacity of the walls.
3. The conclusion that "the failure of well-designed steel plate walls occurred at the column base or at the beam-column

connections" was used to clarify the conclusion that "the local fracture of the infill plates did not significantly affect the overall strength and deformation capacity of the system." This conclusion implies that the well-designed steel plate walls with thin plates do not fail early by the fracture of the infill plates, and they consequently show excellent strength and deformation capacities until the frame members finally fail at ultimate loading.

4. The discussers emphasized the importance of the plate-frame interaction in the design of steel plate walls. As previously mentioned, the discussed study focused on the investigation of the behavior of thin wall plate walls. The plate-frame interaction was considered in Eq. (1) in the discussed paper for the evaluation of the shear capacity of the walls.
5. The discussers mentioned that in all the specimens analyzed by the discussers, the frames became plastic in advance of the plates. The plastic mechanism of steel plate walls can vary according to their configuration, properties, and boundary and loading conditions (Fig. 11 of the paper). However, to maximize the ductility of multiple-story walls with thin infill plates, the walls should be designed to show shear-dominated behavior, as shown in Fig. 11(a) in the paper. For this purpose, the infill plates in multiple stories should yield in shear in advance of yielding of the columns. The columns should be designed with compact sections for the combined axial force and bending moment developed by the tension-field action of the infill plates.
6. Finally, the discussers suggested that the first and second load-displacement curves are presented for better judgment of the results. Though the first and second load-displacement curves were not clearly presented in the figures shown in the paper, the initial yield points and yield points of the test specimens were given in Table 3 in the paper. The yield displacement and stiffness can be used to understand the behavior of the specimens at early loading.