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Use of vertical shear-links in eccentrically braced frames

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ABSTRACT: The excellent ductility and stiffness of eccentrically braced frames using shear links have made these structural systems an effective alternative for both moment-resisting frames as well as concentrically braced structures. In common practice horizontal shear links are located either at the middle or at the ends of the beam sections. However, large deformations of the shear links in those beams must be accepted under severe earthquakes. In those instances in which the beams have to remain elastic, vertical shear links should be designed underneath the beams thereby transferring the region of plastic deformations to locations where they are tolerable and post-earthquake repair or replacement of damaged parts is easier. Practical design considerations as well as results of stability calculations for an actual case are presented and discussed.

1 INTRODUCTION

Eccentrically braced frames (EBF's) using shear links in the beams have been succesfully investigated and developed by Popov (1987) and others at the University of California at Berkeley since the mid-seventies. Numerous structures have been built using EBF's. Also, current research studies by Bouwkamp (1991) have focussed on horizontal shear links designed as composite T-beam sections with infilled reinforced concrete placed between the flanges on either side of the steel section web. However, horizontal shear links as shown in Fig. 1 cannot be used if the beams have to remain elastic. This may be case e.g. in industrial buildings where extreme heavy equipment loads have to be supported.

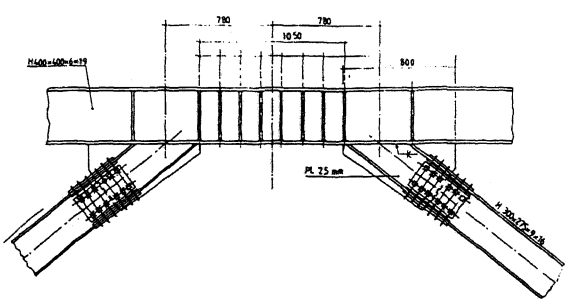

Fig. 1 Typical design of horizontal shear link

In those cases vertical shear links placed below the beams may be used instead as demonstrated by the following application.

2 PRACTICAL PROBLEM

For the earthquake resistant design of an approximately 70 m high reactor tower of a direct iron-ore reduction plant the problem of developing a ductile bracing system did arise. The use of the structure required access from the side to install and maintain the heavy operating equipment located on the platforms thus making conventional X-bracing operationally unacceptable. Hence, to overcome this structural restriction, inverted V-bracings would have been an operationally desirable solution. However, using such a brace arrangement in a concentric truss system would not ensure sufficient ductility since buckling of the compression diagonals would precede yielding of the tension-diagonals.

Also, constructing an EBF using eccentric inverted V-bracing and beam shear-links for all stories was not possible since at several of the floors the girder grid system had to carry very large vertical loads and the operational requirements stipulated that the girders were required to remain elastic even under severe earthquake conditions. Furthermore, it was required to minimize the restraints of temperature induced deformations and differential shortening of the tower columns. Since this goal is impossible to achieve with conventional horizontal shear-links, vertical shear-links connected to the inverted-V-braces have been chosen for this project.

2.1 General design considerations for vertical shear links

The design of the individual shear links follows basically the recommendations for horizontal shear links as presented by Ricles (1988) and stipulated in the Recommended Lateral Force Requirements of the SEAOC (1988). Applying these recommendations a vertical shear link can be designed as a vertically oriented beam-stub welded to the bottom flange of the locally web-stiffened horizontal girder. The bottom of the vertical shear link has a horizontal end plate to which typically a vertically oriented gusset plate would be welded for subsequent connection (bolted) of the inverted V-braces. However, this fully interwelded design solution would not meet the no-restraint requirement for differential vertical deformations of the columns and/or diagonals. Hence, a gap below the stub endplate and the rest of the inverted V-brace connection had to be introduced. The basic gap arrangement, being it for a double shear-link design, is shown in Fig. 2.

It should be mentioned, that depending on the required web thickness in relation to the depth of the web, the SEAOC recommendations would result in an unpractical web stiffener spacing in some cases. This is particularly problematic for shear links subjected to small shear forces. Considering the overall design requirements (loads) and the code stipulations, in certain locations it was found to be unpractical to design the shear link as a single element. Hence, a design arrangement with a pair of shear links as shown in Fig. 2 has been developed for certain locations.

In such designs the two shear links are coupled by a connecting plate in order to obtain an almost symmetric alternating load-history exposure.ingle shear links instead of shear forces of constant sign. Otherwise large plastic deformations would remain after the earthquake and the gap between the links would enlarge drastically.

2.2 Lateral stability of vertical shear links

As only very limited information on vertical shear links is available in the literature, questions especially with regard to the lateral stability had to be addressed. In tests with horizontal shear-links, as reported by Popov (1988), lateral buckling has been observed when large deformations occur and insufficient lateral restraint is provided. These observations are reflected in the SEAOC recommendations.

However, the ductility level at which lateral buckling occurs might be higher than the required one. In this sense, it can be expected that cantilevering vertical shear-links behave in the same manner as horizontal shear-links. Thus, the question

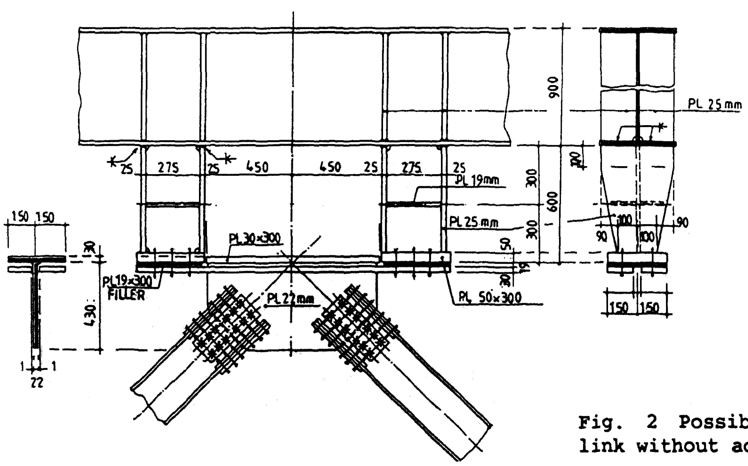

Fig. 2 Possible layout of vertical shear link without additional bracing

is whether vertical shear-links as shown in Fig. 2 may be designed without additional lateral bracing or not.

2.3 Modeling

In order to assess the stability of vertical shear links, a simplified model of a single link has been investigated taking into account geometrical and physical nonlinearities. For this analysis, the program STABET, developed by Pauli (1990), has been used. This program offers the possibilities to investigate combined bending and torsional instability problems and permits introducing several nonlinear material laws. However, STABET only contains beam elements and no plate or membrane elements. Hence, an equivalent strut and tie bar-element model was formulated to capture the most important effects (see Fig. 3).

The web of the shear link has been modeled by diagonal struts while each of the flanges has been modeled by a line of beam elements.

In addition to these beam elements which represent the location and actual geometric properties of the flanges, a second line located on the outside has been introduced in the model in order to capture the fact that, in reality, the middle of each flange is connected continuously with the web thus

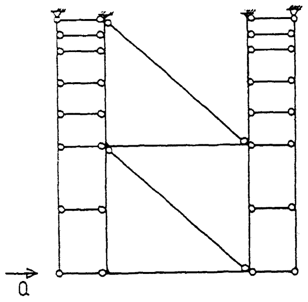

Fig. 3 Strut-and-tie model for single vertical shear link

stiffening the flange. In order to model this stiffening effect, the exterior line of beam elements is introduced and connected to the interior line by effectively rigid truss elements ensuring the same horizontal deflections in both lines. The exterior line elements have extreme bending stiffnesses.

Physically, this model can capture the web yielding and ascertain that the intersection between web and flanges remains virtually straight except there where the shear-link flanges have been welded to the girder flange. At those locations plastic hinges must develop in the flanges in order to enable shear yielding of the web (see Fig. 4). Once the plastic hinges have been formed, local plate-bending of the flanges occur.

The spacing of the connecting truss elements has been chosen in such a way, that local flange buckling may be captured approximately in the model. Although, the model has been developed with care, it is not claimed that the local buckling behaviour of the flanges will be captured absolutely correct. The model should be regarded as an engineering approximation capable to assess the overall shear-link behavior with sufficient accuracy.

As already mentioned above, the shear panel is represented by diagonal truss elements. The initial calculations showed, however, that it is advisable to use tension diagonals only. Modelling the compression diagonal such that this element will capture the nonlinear web buckling correctly would have been too complex and of little practical value for this engineering assessment. In fact, considering that premature web buckling can be excluded due to the narrow web spacing (as stipulated by the specific design rules concerning web thickness, web depth and stiffener spacing), this approach seems to be justified.

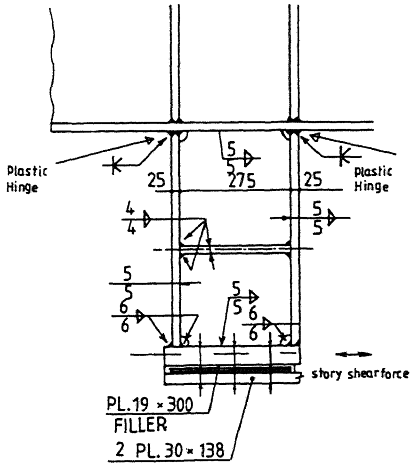

Fig. 4 Shear-link details

2.4 Constitutive relations for materials

Experiments with typical horizontal shear-links as well as code provisions (e.g. SEAOC) show or assume an effective stress increase due to strain hardening of up to 50% (i.e. max $Q = 1.5 * Q_{pl}$). Thus, it is necessary to study the shear-link itself, considering the strain hardening of the web, as well as to investigate the capacity of adjacent structural elements under the effect of the possible load increase due to the web strain hardening..

However, part of the possible increase in plastic shear-capacity is associated with the bending capacity of the flanges of the shear-link. Hence, the maximum shear stress in the web should not be considered to increase 50 % over the elastic limit but only to a somewhat smaller value. In order to capture this effect, the constitutive model used for the steel-diagonal, representing the web, has been reduced to about 83.3 % of the above value (or 1.25).

Since the shear in the web is represented by diagonals, the ultimate stresses had to be reduced in order to be compatible with the von-Mises-rule. For an equivalent truss system adjusted to the same stiffness of the web-panel, a factor of 0.75 resulted for this case. Hence, the nominal yield stress to be used for the digonals amounted to

$$f_y = 24\ KN/cm^2 * 0.75 = 18\ KN/cm^2$$

whereas the ultimate stress considering strain hardening was

$$f_u = 24\ KN/cm^2 * 1.5 * 0.833 * 0.75 = $$
$$= 22.5\ KN/cm^2.$$

In order to avoid in the analysis problems with lack of convergence in the iteration, an increased nominal yield strain in conjuntion with a reduced value for the elasticity-modulus has been chosen. Thus, in the range of strain which is of interest (i.e. over 0.03), as shown in Fig. 5a, the material is represented with correct stresses as well as with a correct tangent-stiffness. For smaller strains, however, the capacity of the material is underestimated. Considering this aspect, the results of the analysis are expected to be on the conservative side.

2.5 Safety considerations

Because time limitations did not permit to verify the computer model by an experimental study, a certain margin of safety seem to be justified to account for model-uncertainties. In order to fulfill this requirement the calculated ductility should be larger than the required one (which is defined by gamma = 0.06). In addition, it should be proved that in the adjacent elements the stresses do not exceed 0.8 *fy (capacity design criterion, see also requirements given by SEAOC). For a check under increased horizontal loads it is necessary to assume an increased strength of the shear panel (reflecting scatter in actual strength versus nominal minimum strength).

Two calculations with different material-values have been performed, namely:

1.) a calculation with nominal values, adjusted to deliver 1.5 * Qpl for a gamma = 0.06 (a horizontal deformation of 3.60 cm for a 60 cm high shear link), and

2.) a calculation using the same model, but with an assumed 25 % increased strength of the diagonals representing the web panel.

2.6 Results of analytical investigation

The maximum deformation delivered by the first calculation amounted to 4.89 cm. This value (corresponding with gamma = 0.08) reflects a safety margin in ductility which may be considered as sufficient. However, both calculations showed a stability failure after reaching a load-factor of 1.125 (related to 1.5 Qpl). Thus, it was impossible to prove the required additional safety margin of at least 25 % against failure of adjacent elements.

The load-deflection plot in Fig. 5b, depicting the transverse deflections (y-direction), show clearly the lateral instability limit of

$$Q_{kr} = 1.125 * 1.5 * Q_{pl} = 1.69\ Q_{pl}$$

which is independant of the web capacity.

3 CONCLUSIONS

It should be noted that, although the model neglected strain hardening in the flanges, in reality strain hardening can be expected in those regions. This effect might increase the lateral-instability limit. However, it is not common practice to make use of such assumptions.

In connection with the fact, that the investigation only covered a single special shear link and others may be less favourable, it has been proposed to stiffen all vertical shear-links laterally in order to prevent stability-failure.

In order to design the lateral stiffening-elements for the shear links it was suggested to use 1/50 of the shear force as stabilizing force in transverse direction.

Considering tilt angles of 1/150 for the shear-link additional calulations gave lateral stabilisation forces in the order of 1/75 to 1/100 of the shear-link force. Hence, 1/50 should be considered as a conservative assumption when checking the adjacent elements.

Since the check of local instability of

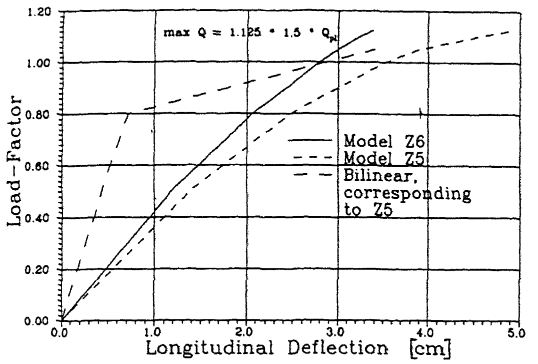

Fig. 5a Load-deflection curves in longitudinal direction

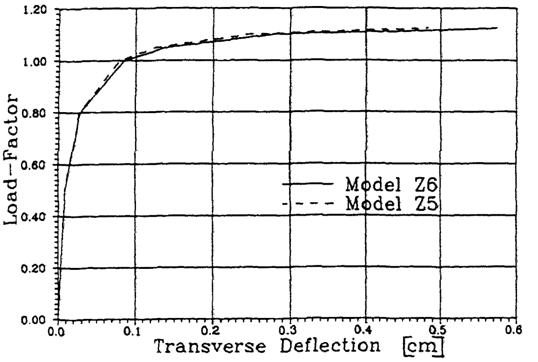

Fig. 5b Load-deflection curves in transverse direction

the shear-link itself is not sufficient to assess the overall stability, a check of the integral system consisting of beam, shear-link and inverted-V-truss is necessary. As far as the shear links are stiffened sufficiently in the lateral direction, a linear-elastic second-order model will be sufficient to check the overall stability of the bracing system including the shear-links.

For this purpose again the program STABET is used since it enables the treatment of bending/torsion-problems of stability-theory.

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