SEISMICALLY LOADED HOLDING DOWN BOLTS

by

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SUMMARY

The investigation described was prompted by recognition of the need to improve understanding of the tensile yielding foundation holding down bolt connection and to resolve whether satisfactory details are being achieved reliably as a result of current design and fabricating practice.

Both the loads and the post-yield axial distortions resulting from earthquake generated movements are determined for holding down bolts from theorectical considerations. Full scale testing of a series of typical bolts was undertaken to establish the actual behaviour characteristics and these are compared with those deemed necessary for satisfactory seismic resistance. Based on these investigations, a rational design procedure is proposed.

BACKGROUND

The practice of restraining column bases and other structural elements which are in contact with concrete footings, by the use of vertical steel bolts is well established. In the simplest case the head of the bolt is embedded in a concrete pad while the shank passes through a suitable hole in a steel plate which is pulled down on to the concrete as the nut is tightened. Experience has shown that adequate anchorage and bolt strength can readily be provided by simple design procedures, so that uplift and horizontal forces may be resisted satisfactorily in most loading situations. However, in the case of earthquake generated loads certain special considerations have to be appreciated. Most seismic resistant structures are intended to resist small earthquake movements elastically but to undergo some post-elastic response without collapse in more severe events. Consequently, it could be expected that unless holding down bolt groups are to be designed so that their elastic limit strength is significantly greater than that of the corresponding column - which requirement usually results in an impractically large and complex base plate details - many holding down bolts will be stressed into the yielding range in a major earthquake.

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In fact normal practice appears to have been to design holding down bolts for only nominal forces and then to either accept or ignore that yielding of the bolts will occur in a large seismic disturbance. Investigating teams reporting on the damage resulting from major earthquakes have substantiated that permanent stretching of holding down bolts takes place. It seems very probable that the energy absorption and reduced stiffness conferred on the structural system by the graceful degradation of the holding down details has, in many cases, contributed very significantly to the ability of the structure to survive the earthquake.

A typical column base in which holding down bolts are employed is shown in Fig. 1. In situations where failure of the bolts would result in catastrophic collapse of the structure special provision may be made to ensure that a significant length of unrestrained bolt is available to accomodate tensile yielding deformation. The vulnerability of vertical cantilever structural systems in seismic areas has resulted in details such as that shown in Fig. 2 being used at the base of steel chimney stacks (1). It is usually not difficult to provide a reasonable length of bolt shank clear of the footing concrete. It is more difficult to ensure that the structural configuration and material properties will permit the development of satisfactory ductile yielding characteristics under severe loading conditions.

FAILURE MODES

Designers of civil engineering structures traditionally have chosen bolted connection configurations in which the loads are resisted by shear rather than tension in the bolts. This preference probably stems from considerations of monotonic loading and the relative ease of ensuring that the threaded zone is not critical when bolts are used in a shearing mode. However, in situations where significant ductile capabilities are required of the bolt system it is improbable that the optimum overall ductile performance can be achieved when an unavoidable load path possesses the brittle characteristics of a shearing failure mode. Although in most situations holding down bolts will be required to resist some horizontal forces by shear resistance, when overall lateral loading on the structure is of major concern the critical loads sustained by bolts in the base of frame columns or of vertical cantilever stack-like systems is likely to be axial tension.

If a reasonably ductile (medium carbon) plain steel bar of a diameter similar to a holding down bolt is stretched in tension one can expect a significant overall extension to occur before fracture. This can be considered to be provided by the progressive development of several yield zones along the bar; as the first one forms, strain hardening will increase the effective strength locally thereby prompting another critically stressed portion of the bar to yield. Several such strain hardened portions may develop before the incipient tendency to "neck" to failure causes the bar to break. Appreciation of the changes in stress distribution caused by the provision of threads on such a bar has led to the development of special purpose bolts including those with upset threads or turned down lengths. Since the stress in the first loaded turn of the threads tends to be critical, unengaged thread between the nut and the bolt shank serves to reduce the local concentration of stress although recognition is necessary of the anomaly which exists between the improvement of static strength, brought about by sharp stress transitions which provide significant plastic restraint, and the requirement for smooth transitions, particularly at the thread run out position, if good dynamic performance is to be achieved (2).

Civil engineering holding down bolts are almost invariably formed by cutting threads onto an otherwise plain bar and hence consideration of other (special) configurations appears unrealistic. Satisfactory yielding in the shank of holding down bolts as a result of earthquake action has been observed (1,3) although apparently premature failure of bridge restraining bolts under seismic loading has also been noted (4).

It is evident that a designer should be in a position to specify what material properties, geometric configurations and manufacturing methods will lead to a reliable energy absorbing performance for holding down bolts subject to post-yield action.

TESTS UNDERTAKEN

If holding down bolts are formed from typically available materials such as those encompassed by properties of minimum yield 245 M Pa. with ultimate 430 to 510 M Pa. or of minimum yield 275 M Pa. with ultimate 380 to 520 M Pa. satisfactory ductile behaviour may be expected. However, when designers and suppliers are not both aware of the importance of compliance with stress limit bounds, problems are likely to arise. The fact that a minimum yield stress is frequently specified for steels and that this is often significantly exceeded by manufacturers is of little advantage to earthquake engineers whose requirements include that of large ductility coupled with a basic minimum strength. The desirability of establishing that typical locally produced holding down bolts do exhibit suitable tensile yielding characteristics prompted the testing described below.

In an attempt to simulate representative holding down bolts, a 44.45 mm diameter round bar, purchased from a steel supplier as "mild steel" was cut into 1400 mm lengths and each had BSW threads cut on both ends using a workshop lathe. The rods were loaded in tension by pushing the ends apart using twin 1000 kN jacks reacting against a concrete floor, at the bottom end, and a specially fabricated cross head at the top end (see Fig. 3). Load cells were used to monitor the applied axial forces and the bolt extension measured using a 50 mm travel dial gauge reading to 0.01 mm. Each of the several bolts tested failed in the threaded zone adjacent to the nut with about 2% total elongation, notwithstanding the fact that the cross sectional area of the thread root exceeded 70% of the shank cross sectional area. A typical load/deflection curve is shown in Fig. 4 and confirmation of the unsuitability of the material for the chosen purpose was obtained when a test specimen was loaded in a Tensile Testing Machine, no yield plateau being established and only a 11% elongation over the turned down section being obtained before fracture. It was subsequently confirmed that this batch of bolts had been manufactured from a free cutting "bright" steel much favoured by jobbing workshops for its ease of turning in bolt manufacture and similar processes.

Subsequently, a further series of tests were undertaken on similar bolts but manufactured from 40 mm diameter bars complying with the specifications appropriate to FY40 reinforcing steel. Several of the test bolts were turned down to 38 mm diameter and had die cut threads added to take 1.5 inch Whitworth nuts at each end, whereas another set had 10 t.p.i. U.N.F. threads rolled on to them as is undertaken in the production of Macalloy prestressing bars. A typical load/deformation trace for a standard test specimen tested in a Tensile Loading Machine is shown in Fig. 5 The plot obtained from testing a typical die cut thread bolt is shown in Fig. 6 and that provided by a representative rolled thread specimen in Fig. 7

The die cut thread anchor bolts each displayed a definite yield point at 275 M Pa. in the threaded section followed by strain hardening, with subsequent yielding in the shank. Fracture occurred in each case in the threaded section adjacent to the nut at an elongation of about 7% of the total bolt length. The rolled thread specimens each exhibited a yield plateau at about 275 M Pa., progressive yielding along the length of the bolt, necking leading to eventual fracture in the central part of the shank and total bolt extension of the order of 25% of the overall length.

DESIGN CONSIDERATIONS

In designing holding down bolts base shears and base moments may be obtained using the appropriate loadings code but recognition must be made of the fact that the magnitude of seismic loads specified in normal mode, response spectra based building codes reflects the situation applicable in most building structures, namely that the loads predicted on the basis of a strictly elastic response are reduced significantly when deriving design values in acknowledgement of the load carrying capacity of the other than primary structure and the fact that non-linear response is expected in moderate to large earthquakes. Some codes incorporate additional factors by which the basic seismic coefficient has to be increased in cases where inadequate redundancies are deemed to exist and where even the onset of yield is likely to lead to catastrophic failure.

Thus the initial selection of the number of size of holding down bolts may reasonably be made on the basis of design loads derived from building codes - probably with a significant factor of safety included to compensate for the likelihood that not all bolts will share equally in the load resistance and for the somewhat indeterminate superposition of shear and axial stresses in any particular bolt. If a rigid seating ring at the base of a chimney stack is assumed together with a distribution of axial load in each bolt in proportion to the horizontal distance of the bolt to the axis about which rotation is taking place, the load in the worst loaded bolt may be assessed.

An indication of the yield distortion capability required in holding down bolts may be obtained from considerations of the energy dissipated by plastic yielding as a proportion of the total seismic input energy. In the approach suggested by Housner (4) the maximum energy E attained by an oscillator subject to seismic excitation is estimated using the expression

$$E = \frac{1}{2} M S_{v,n}^2$$

where M is the mass of the oscillator
and $S_{v,n}$ is the velocity spectrum ordinate for damping n,

assuming the velocity spectrum to consist of a straight horizontal line.

If, for the purpose of an example, we consider a chimney of 30 m height and 50,000 kg mass then $E \doteq 40{,}000$ Joules for a velocity spectrum similar to that obtained from the Poicoma Dam (1971) record, assuming 2% critical damping. If thirty symmetrically spaced holding down bolts restrain the chimney (as in Fig. 2) and the simplifying assumption of a rigid base ring pivoting about one edge is made, then the worst stretched bolt will have to absorb of the order of 4,000 Joules by post-elastic deformation, a performance clearly within the capability of a bolt possessing the load/extension characteristics depicted in Fig. 6. The use of the velocity spectrum appropriate to the El Centro 1940 N/S record might be considered to be a more appropriate choice by a designer in which the case the worst stretched bolt would have to absorb approximately a quarter of the energy estimated above on the basis of a symmetric allocation of energy absorption. As this distribution is unlikely to be achieved in practice, provision for extensions of magnitude several times that predicted by the use of such a simple model could be judged prudent.

POSSIBLE DESIGN PROCEDURE

Based on the considerations outlined above a rational design procedure for holding down bolts may be devised. For clarity reference will be made to a particular application, namely a steel chimney of 30 m height, 3 m diameter, 50,000 kg mass and 0.7 second fundamental period. Suppose 24 bolts are to be used, symmetrically spaced around the base on a circle of 3.2 m diameter.

An initial bolt size may be determined using a typical building code calling for a seismic coefficient of (say) 0.2 g. Shear stresses will be relatively insignificant and a working stress axial load design approach will result in a bolt diameter of around 25 mm being established although recognition of the limitations of this procedure will probably prompt the selection of a bolt size considerably larger than this for the actual detail finally specified. Having made a choice of bolt size, the designer should next calculate the likely post-yield axial deformation requirements with the object of satisfying himself that the particular bolts finally selected are capable of enduring such extensions without premature fracture. Testing of representative samples may be necessary to provide the required reassurance. In the case of the chimney considered here, application of Housner's procedure (3) (with $S_v = 0.625$ m/s) leads to a requirement for 8,000 Joules to be absorbed by bolt yielding and if, in the worst case, four bolts are assumed to have to provide this energy absorption capability, then assuming the bolt characteristics presented in Fig. 6 as appropriate, the necessary post-elastic extensions of around 10 mm could be accommodated satisfactorily.

In the event of the bolts chosen on the basis of purely elastic design considerations being found inadequate when the post-yield capabilities are examined, appropriate re-design should be undertaken until satisfactory compliance with both sets of criteria is achieved.

CONCLUSION

A fundemental problem facing earthquake engineers, namely the desirability of achieving adequate strength coupled with large ductility rather than excessive strength coupled with limited ductility, is of particular concern in holding down bolt design. Also the impracticability of providing a preferable configuration (i.e. upset rolled threads) in normal civil engineering construction results in the almost universal use of cut-thread bolts. Adequate axial yielding may be achieved, even though failure must almost invariably occur in the threaded zone, provided that appropriate care is exercised by the designer in his choice of materials and details. Considerable effort should be expended at the supervision stage to ensure that installed holding down bolts will possess the capability to behave as intended under severe loading conditions.

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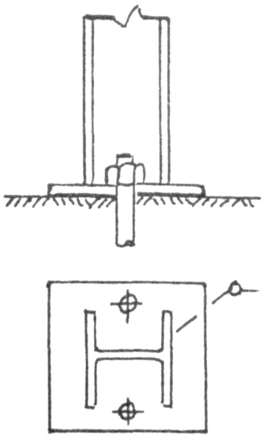

Fig. 1 Holding down bolts in column baseplate

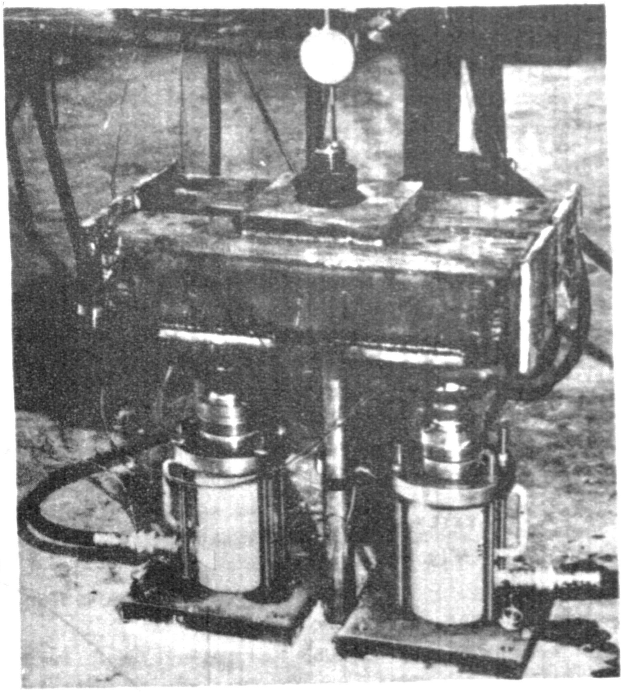

Fig. 3 Test Rig

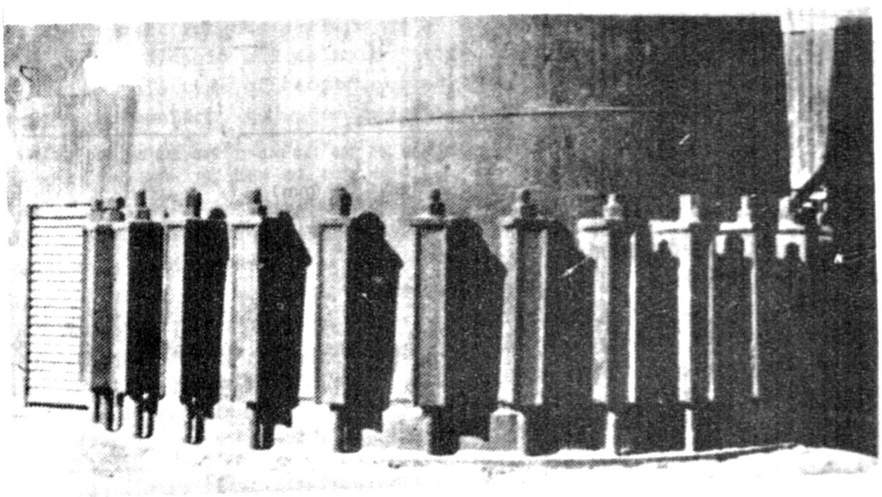

Fig. 2 Base of chimney

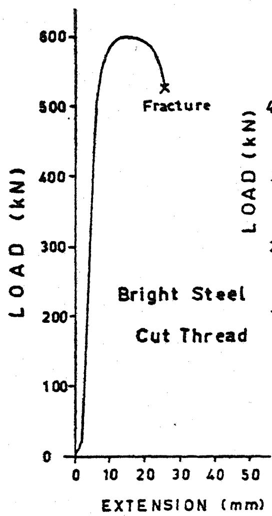

Fig. 4

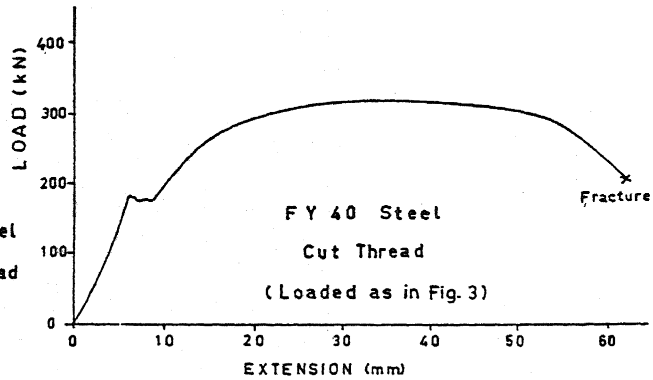

Fig. 6

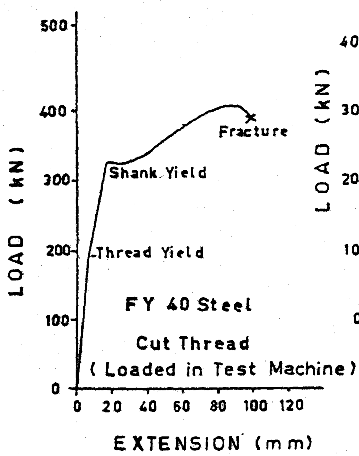

Fig. 5

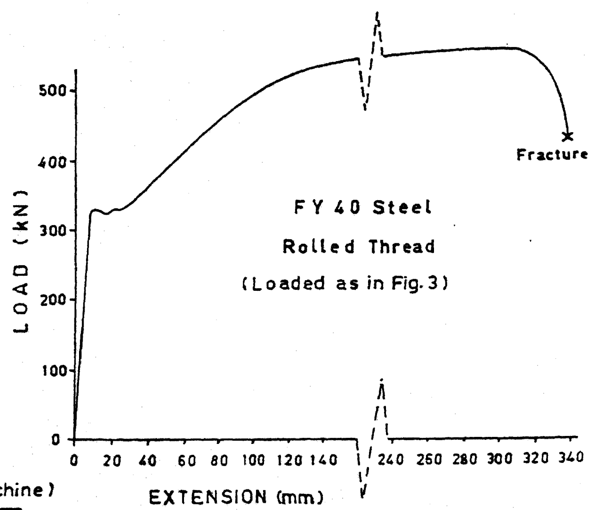

Fig. 7