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SHAKE-TABLE TESTING AND SEISMIC PERFORMANCE EVALUATION OF BRACING MEMBERS

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SUMMARY

This paper deals with the seismic behaviour of bracing members. An experimental investigation involving shake table tests on idealised concentrically braced single-storey frames is decribed. The testing arrangement, specimen configuration and choice of seismic input are summarised. Typical results are presented and the main experimental observations are highlighted. Elastic tests were first conducted to examine basic dynamic characteristics, followed by large amplitude seismic tests to assess the inelastic behaviour. The frames incorporated hollow steel bracing members with three different square and rectangular cross-sections sizes in order to provide a range of section and member slenderness. In addition, a number of hollow members were in-filled with mortar for comparison purposes.

An assessment of a number of key design aspects is undertaken on the basis of experimental observations and findings as well as consideration of underlying response mechanisms. The main differences in the interpretation and simplification of response within codes of practice are discussed, particularly in terms of dealing with the brace buckling in compression. The implications of these differences on the overall behaviour, primarily in terms of the expected lateral overstrength as well as the level of inelastic ductility demand on the bracing members, are pointed out.

The limiting criterion for the bracing members was illustrated in some of the tests, in which brace failure occurred through fracture of the cross-section following the on-set of local buckling. Depending on the section and member slenderness level, beneficial effects may be realised by in-filling the hollow braces which could delay or inhibit local buckling. The tests also indicated that bracing members with slenderness exceeding the limits imposed by some seismic codes demonstrated generally satisfactory performance. Despite their lower energy dissipation capabilities, there are several practical and design advantages that merit their utilisation.

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INTRODUCTION

Although concentrically braced frames may represent an effective structural form for providing lateral stiffness and strength under earthquake loading, the inelastic performance of diagonal bracing members require careful examination. In a severe seismic event, bracing members experience inelastic deformations in cyclic tension beyond yield and compression into the post-buckling range. The cyclic behaviour of bracing members has therefore been investigated by several researchers [e.g. 1-9]. Most available investigations are however based on quasi-static cyclic tests conducted under pre-defined displacement histories for specific ranges of member slenderness, with comparatively fewer studies performed under realistic dynamic loading conditions.

In terms of design procedures, seismic codes of practice [e.g. 10-12] typically adopt the general philosophy of capacity design. In the case of concentrically-braced frames, capacity design normally implies the use of diagonal bracing members as the main dissipative elements, whilst providing adequate overstrength factors for other frame members and components to ensure compliance with the selected failure mode. Whereas seismic codes largely agree on this overall philosophy, there are notable differences in the design of concentrically braced frames [13]. Some of these disparities are related to key design approaches, whilst others are associated with geometric and dimensional limitations. Such inconsistencies need to be clearly identified, and their implications on capacity design should be appropriately considered.

In this paper, an experimental study into the seismic behaviour of bracing members is summarized. Several shake-table tests, performed to investigate the dynamic response under realistic seismic conditions, are described. In each test, the bracing members were arranged such that they represent an idealization of one storey within a typical form of concentrically braced frame in which the load sharing between the tension and compression braces is accounted for. The tests were undertaken as part of a European project involving experimental, analytical and design studies.

Following a brief description of the experimental arrangement used for the shake-table tests, selected results are presented and discussed. These involve tests on diagonal bracing members with square or rectangular hollow cold-formed steel cross-sections incorporating different section and member slenderness. Additionally, reference is made to a number of tests in which the tubular diagonal members were in-filled with mortar in order to examine the comparative behaviour between steel and composite counterparts. In the light of experimental observations, and within the context of capacity design, a number of key response parameters and design considerations are highlighted and discussed.

EXPERIMENTAL PROCEDURES

The shake table tests were carried out at the Laboratory for Earthquake Engineering of the National Technical University of Athens (NTUA). With due consideration of the objectives of the experimental programme coupled with the table characteristics and limits, the arrangement depicted in Figure 1 was adopted. The set-up consisted of an assembly of four relatively rigid columns, nominally-pinned at both ends in-plane and adequately-braced in the other direction. At the top, a grillage of beams formed a rigid diaphragm to which the test masses were attached. The total mass above the columns, including those of the steel beams, amounted to approximately 10 tonnes. A more comprehensive account of the experimental arrangement and details is presented elsewhere [14].

In each test, two brace specimens were rigidly connected at their top ends to the transverse beams, and at their lower ends to the platform of the shake-table. Importantly, the two braces were not connected at mid-length, but were sufficiently separated in plan in order to avoid contact in the case of out of plane buckling. This arrangement accounts for the interaction in terms of load sharing between the braces within a typical storey in a general form of a concentrically braced frame. However, it does not represent the additional considerations related to the assessment of buckling lengths in inter-connected brace configurations.

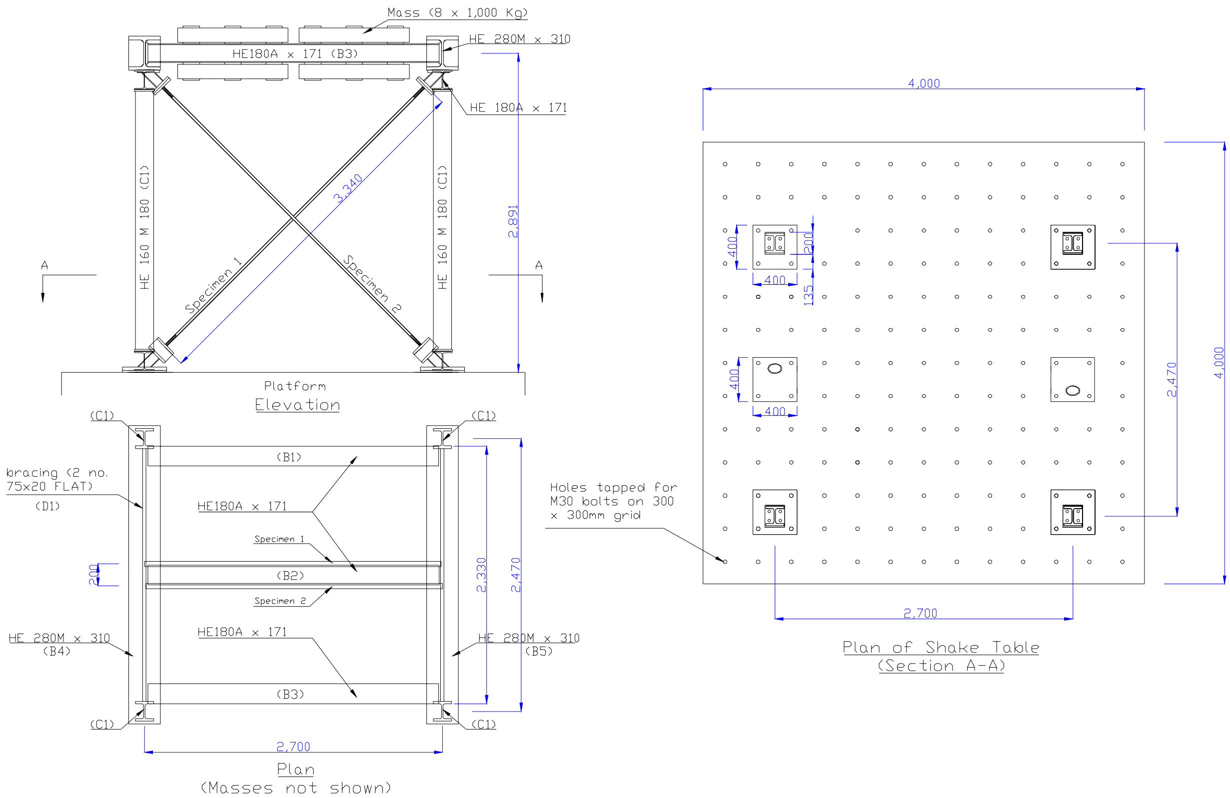

Figure 1 Experimental arrangement for shake-table tests

The instrumentation included load cells to measure the axial load in the braces, bi-directional accelerometers at various locations on the mass, cable-extension transducers to measure lateral deformation at the table and mass levels against external reference points, and displacement transducers along the length of the braces to measure axial deformations. Several strain gauges were also used on the brace specimens. Additionally, other gauges were permanently placed on the various rig members to verify that these components behaved in an elastic and relatively rigid manner, in accordance with the predictions of analytical and design checks. This ensured that the response was dominated by the behaviour of the bracing members as intended, which is also useful for facilitating comparison with analytical studies.

The size and details of specimens were selected to cover a range of slenderness as well as to satisfy the experimental constraints. Two cold-formed steel square hollow sizes, namely 20x20x2 mm and 40x40x2.5 mm, and one rectangular hollow size 50x25x2.5 mm, were utilized. The end detail of each member was prepared by welding a stiffener plate which passed through the centre of the hollow section. End plates were also welded to the specimens to facilitate connection to a load cell on one side and the transverse beam at the other end. Based on this design, the braced had a clear length of 3.05 m with nominally rigid end conditions. The initial imperfections along the length were measured, and the average amplitude was found to be about 0.2%.

Some of the specimens tested within this experimental programme are selected and described in Table I [14]. It should be noted that in cold formed elements, cold-working effects can lead to yield strengths, which are higher than that obtained on the basis of coupon tests. Some codes, e.g. Eurocode 3 [15], therefore provide expressions for the average material strengths that can be applied to the cross-section. Alternatively, the yield strength can be determined directly from tensile tests on the full section. In this investigation, both coupon and section tensile tests were conducted. The average strengths obtained from section tests was about 345 MPa for yield and approximately 395 MPa for ultimate. On average, these values were about 10% higher than those from coupon specimens. In the case of composite specimens, the hollow members were in-filled with a mortar material of cement and fine aggregate, with an average compressive and tensile splitting strength of about 24 and 2.5 MPa, respectively. Further details related to the material properties and specimen details can be found elsewhere [14].

Table I Summary of inelastic tests

Specimen Reference	Section Size	Slenderness *	Input Type	Scaling Factor	N_{max}/N_{pl}
H20A	20x20x2.0	2.8	Sine ramp	-	1.18
H20B	20x20x2.0	2.8	Synthetic	140%	1.10
H20C	20x20x2.0	2.8	Natural	140%	1.12
F20C	20x20x2.0	2.8	Natural	140%	1.18
H50A	50x25x2.5	2.0	Natural	250%	1.07
F50A	50x25x2.5	2.0	Natural	250%	1.08
H40A	40x40x2.5	1.3	Natural	320%	1.09
H40B	40x40x2.5	1.3	Natural	320%	0.98
F40B	40x40x2.5	1.3	Natural	320%	1.01

*approximate estimates based on bare steel section and average material properties

The non-dimensional slenderness $\bar{\lambda}$ for various brace members was within the range of 1.0 to 3.0. For non-slender cross-sections, $\bar{\lambda}$ is defined in EC3 [15] as $(N_{pl}/N_{cr})^{0.5}$, in which N_{pl} and N_{cr} are the plastic section capacity and the elastic (Euler) buckling load, respectively. Assuming average material properties, and nominally-rigid end conditions, the value of $\bar{\lambda}$ is estimated to be about 2.8, 2.0 and 1.3 for steel braces with size 20x20x2.0, 50x25x2.5 and 40x40x2.5, respectively. Clearly, depending on the code, the value of $\bar{\lambda}$ for the in-filled braces would vary from that in the bare steel counterpart. In Eurocode 4 [16], account is taken of the additional strength provided by the concrete in determining N_{pl}, and an equivalent stiffness for the composite section is used in evaluating N_{cr}; this results in an increase in slenderness compared to the above-estimated values of about 15%. As expected, the non-dimensional slenderness has a direct influence on the behaviour of the bracing members as discussed in subsequent parts of this paper.

Prior to performing the inelastic test, each frame was subjected to a series of elastic tests to examine the initial dynamic characteristics. In the elastic range, the input was in the form of a sine-sweep and, for confirmation, a random signal at constant amplitude. Using spectral assessment, the results were then used to estimate the natural frequency, and the critical damping was evaluated from the half-power bandwidth approach. The elastic tests indicated average natural periods of vibration of about 0.26, 0.18 and 0.16 seconds for the frames with 20x20x2.0, 50x25x2.5 and 40x40x2.5 braces, respectively. The typical critical damping ratio obtained in the elastic tests was approximately 3.1%. No notable difference was observed between the periods of the steel and composite braces, with the natural frequency being more sensitive to the level of initial imperfections, as expected.

For the inelastic tests, most of the frames were subjected to a scaled acceleration history from the Imperial Valley record of the El Centro earthquake. The original record, with a peak ground acceleration of about 0.34g was appropriately scaled for each frame depending on the expected strength, as indicated in Table I. For comparison purposes, Frame H20B was subjected to an artificially-generated record whose response spectrum closely resembles the design spectrum of EC8 [10]. The peak ground acceleration, also assumed to be 0.34g as in the El Centro excitation, was amplified accordingly. Moreover, for Frame H20A, a sine-ramp was employed for comparison. It should be noted that Test H40B was nominally identical, in terms of brace specimens and input excitation, to H40A. This was undertaken to confirm the specific features of response obtained for this particular size of brace, as discussed below. Prior to each test, a low amplitude version of the seismic record used was applied in order to confirm the range of magnification, which was typically in the range of 2.5-3.2 under the earthquake excitations.

RESULTS AND OBSERVATIONS

Before applying earthquake excitations into the inelastic range, it was useful firstly to examine the response of one of the frames under an idealised input representing a harmonic motion of constant frequency and gradually increasing amplitude. As shown in Figure 2, a sine ramp increasing to about 1.0g was applied at a constant frequency representing approximately 80% of the estimated natural frequency of the frame. The figure also depicts the acceleration response which is limited to about 0.4g in accordance with the capacity of the frame as determined by the braces.

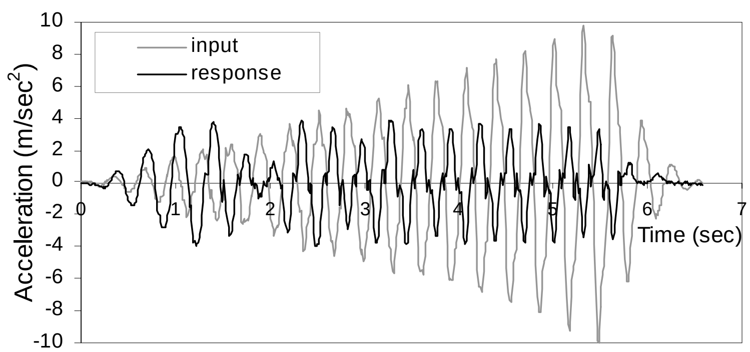

Figure 2 - Input and response acceleration under sine-ramp

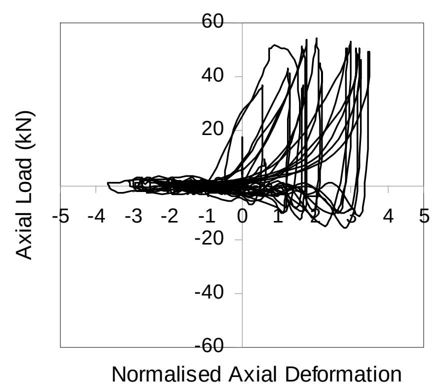

Figure 3 – Typical brace behaviour

In order to illustrate local brace behaviour, Figure 3 depicts the hysteretic response of one of the braces noting that the axial deformation is normalised by the estimated yield value. The figure demonstrates typical cyclic behaviour characterized by tensile yielding and compressive buckling although the high slenderness of the brace implies that the recovery of buckling on tensile-reversal reduces the level of yielding even at relatively large lateral deformations. It should also be noted that, due to the idealised nature of the input, the overall frame behaviour was highly symmetric. This implied that the measured inelastic deformations in the two braces were largely similar, resulting in consistent ductility demand in both directions of the response.

For comparison purposes, H20B and H20C were subjected to synthetic and natural records, respectively. For example, the input and response accelerations for H20C are shown in Figure 4. The main features of the hysteretic behaviour were similar to that observed in the sine-ramp test, but due to the more random nature of earthquake excitations asymmetric effects were clearly observed in the response of the two braces. This is illustrated in Figure 5 which depicts the lateral base shear against the normalised percentage drift for H20C tested under a scaled El Centro excitation. As in other 20x20x2 frames, the overall response had a pinched shape, which is a feature of frames with slender braces, leading to relatively low stiffness near the displacement-reversal point. Although this results in relatively sudden loading-unloading effects, their extent was not as significant as anticipated and gradually reduced with larger deformation amplitudes. In all the test frames employing 20x20x2 braces, no sign of local buckling was observed in either diagonal member.

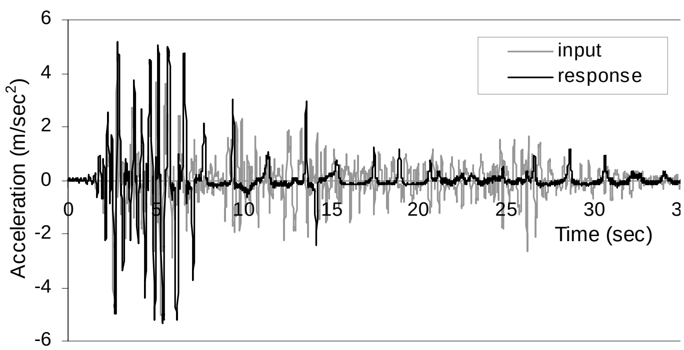

Figure 4 – Input and response acceleration of H20C

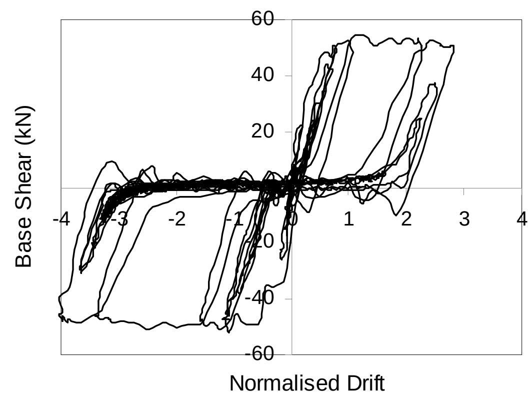

Figure 5 – Base shear response of H20C

The frames with 50x50x2.5 braces were subjected to the same El Centro record but with a scaling factor of 250% in accordance with the expected higher tensile capacity. The peak response acceleration for H50A exceeded 0.9g reflecting the overall lateral capacity of the frames as determined by the braces. The behaviour was similar in nature to that obtained for other braces, but with relatively more stable hysteretic response and less sudden buckling effects in comparison with the 20x20x2 braces. Examination of the steel braces following the tests revealed the presence of slight local buckling at mid-length as well as at the rigid ends.

The scaling factor applied to the seismic record was increased to about 320% in the case of the 40x40x2.5 frames in accordance with the expected frame capacity. For H40A, the peak response approached 1.2g in agreement with the actual lateral capacity of the frame. It should be noted however that, unlike in other tests, the table acceleration did not closely follow the intended input excitation, but was substantially exceeded at two instances corresponding to the peaks occurring after approximately 3 and 6 seconds. This was attributed to interaction effects between the shake-table and the relatively stiff frame tested in this case.

Figure 6 depicts the base shear versus normalized drift for H40A, which indicates relatively more stable hysteretic response in comparison with frames incorporating other braces sizes by virtue of the lower member slenderness. The loading/unloading behaviour was also considerably more gradual in nature, compared to other models. Nevertheless, local buckling had a significant influence on the behaviour and eventually resulted in full failure of the members. At an early stage of the response, around approximately 4 seconds, local buckling became visible at mid-length and close to the ends of both braces. After about 10 seconds, cracks started to open widely at these locations in both braces, most clearly at mid-length.

With further cycles, albeit at lower amplitudes, one brace fractured completely at mid-length. The other brace suffered from partial fractures at mid-length as indicated in Figure 7, and near the two ends.

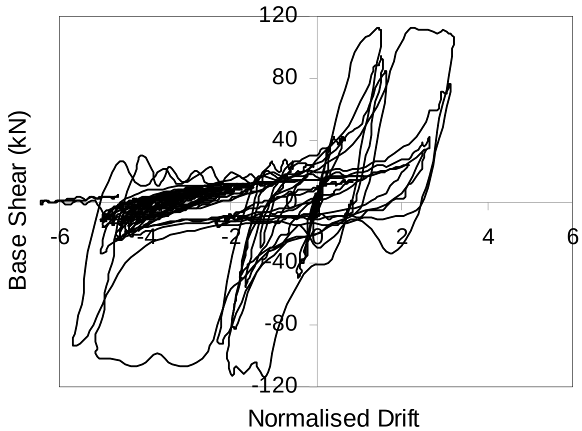

Figure 6 – Base shear response of H40A

Figure 7 – Local buckling and partial fracture

Following the test on Frame H40A, it was decided to subject the other steel frame with 40x40x2.5, H40B, to the same input motion to corroborate the observed table and specimen responses. The performance was very similar to that of Frame H40A, both in terms of peak table and specimen accelerations, as well as the failure mode of the braces involving local buckling in both braces followed by full fracture of one brace and partial fracture of the other.

As noted before, a number of brace specimens were in-filled with mortar for comparison purposes. This was undertaken in-part as additional verification of quasi-static cyclic tests performed within a complementary experimental programme [17] which focused in more detail on the behaviour of in-filled braces in comparison with steel counterparts. Preliminary assessment of the results indicated that in the shake-table tests, the behaviour of in-filled members was largely similar to that of the steel counterparts, in the case of 20x20x2 and 50x25x2.5 braces. Although slight local buckling was observed in the 50x50x2.5 steel braces, this did not appear to have a significant influence on the behaviour. In contrast, the presence of the in-fill played a significant role in the 40x40x2.5 braces. Whilst complete fracture of the braces was initiated by gradual deterioration due to local buckling in the case of steel braces, the composite braces survived the same nominal input excitation without exhibiting any sign of local or member deterioration. Further discussion of these aspects of behaviour is provided below.

DESIGN CONSIDERATIONS

The results obtained from the shake table tests provide necessary information for validating analytical procedures. These would subsequently be employed in wider parametric studies with a view to the assessment and revision of design procedures. In the light of the experimental results and observations, it is useful to comment on some key issues and response parameters, and highlight their role and importance within the context of capacity design procedures for concentrically braced frames.

The actual tensile capacity of the diagonal braces plays a major role in determining the extent of the forces transferred to other frame members and components. However, a number of factors may cause the maximum tensile resistance to exceed its nominal design value. These include higher yield strength, strain hardening and strain rate effects. From the shake-table results, it was evident that an assessment of tensile capacity based on the full section strength obtained from monotonic testing can underestimate the maximum tensile force by up to about 20%, as indicated by the values of N_{max}/N_{pl} in Table I. Clearly, if the assessment is carried out in terms of nominal material properties, significantly higher overstrength factors would be obtained.

To account for all sources of overstrength, codes of practice usually specify strength enhancement coefficients, typically ranging between 10% and 35%, depending on the code. If significant discrepancy is expected between actual and nominal properties, this should be reflected in a higher overstrength coefficient. Moreover, where the evaluation of maximum tensile brace force is necessary in design, partial safety factors for material strength should not be applied. Also, design code procedures for estimating the effect of cold-forming on yield strength should be employed if test data on section tensile resistance are not available.

The buckling capacity and post-buckling residual strength in compression, which are directly related to brace slenderness, obviously have a direct influence on frame behaviour. However, there are several factors that can lead to significant discrepancy from both theoretical and code predictions including actual imperfections, cyclic effects and ductility demand. Moreover, preliminary assessment of the shake-table tests appeared to indicate notable influence from dynamic effects on the compressive strength. Clearly, an adequate assessment of both the upper and lower bounds of buckling as well as post-buckling strengths is important due to its implications for the forces developed in other components as well as the overall seismic response of the frame.

A key aspect influencing seismic response is the global overstrength exhibited by the structure. There are several sources that can introduce overstrength including material or size effects, contribution of non-structural components, etc. Most importantly, overstrength may often be a direct consequence of the simplification of the design approach. In the case of concentrically braced frames, the main simplification in the design procedure is largely related to the treatment of buckling and post-buckling in compression. This issue also represents the most noticeable difference in code provisions. Whereas several codes, such as US guidelines [11] base the design strength on the brace buckling capacity in compression, European practice [10] has contrastingly been based essentially on the brace plastic capacity in tension.

By employing a number of idealisations regarding the maximum expected base shear and that corresponding to the design simplification [13], it is possible to provide a prediction of the frame overstrength on the basis of the two alternative design approaches. This is illustrated in Figure 8 which depicts the storey overstrength of the frame against $\bar{\lambda}$ for both compression and tension based design procedures. As indicated in the figure, for the compression design, the overstrength increases from unity at low slenderness to values exceeding five-fold for relatively slender members. On the other hand, in the case of tension design, the overstength reduces with the increase in slenderness, from a value of two at low slenderness, and becomes relatively insignificant for comparatively large slenderness values.

As noted before, capacity design of concentrically braced frames entails designing structural elements and components, other than the braces, to respond primarily in the elastic range without experiencing yielding or buckling. To achieve this, design forces should be determined with due account taken of overstrength. The maximum forces in the various frame elements would however depend on the location of the component under consideration as well as on the frame configuration. On the basis of the design approach adopted for brace buckling, conservative assessment of loads need to be considered for various design situations. It would probably be impractical for code procedures to assign rules that cover all possible combinations. Consequently, the main assumptions and simplifications should be taken into consideration in order to enable a valid implementation of capacity design procedures.

In addition to the influence of overstrength on the maximum forces that develop in other frame components, it also has a direct implication on the ductility demand. For typical storey arrangements and adopting several idealisations [13], including the assumption that the ductility demand can be related to the behaviour factor ‘q’ through an ‘equal-energy approach’, simple relationships can be derived between ductility demand and slenderness for various behaviour factors. These are represented in Figure 9 on the basis of both tension and compression design approaches for the braces. Initial assessment of the experimental results, coupled with complementary analytical studies, indicates that the curves in Figure 9 provide a reasonable prediction of ductility demand and its relationship with $\bar{\lambda}$ and q.

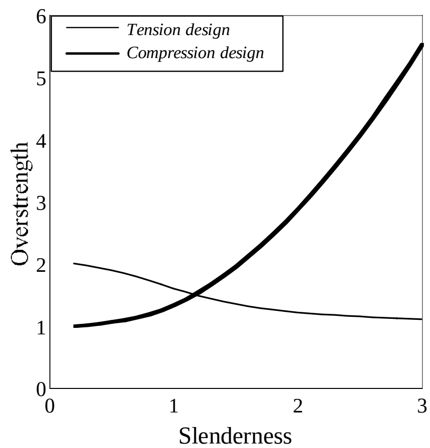

Figure 8 – Assessment of over-strength

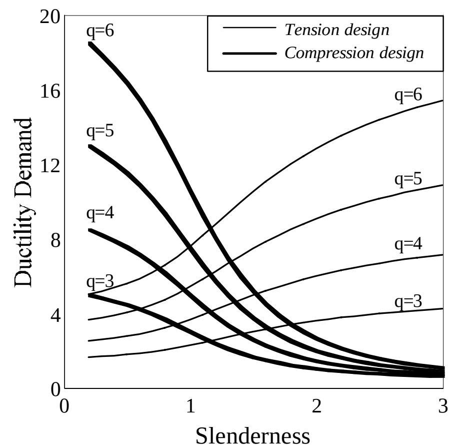

Figure 9 – Simplified prediction of demand

The ultimate failure mode for the hollow bracing members was clearly illustrated in the shake-table tests on frames with 40x40x2.5 braces. High strains typically develop upon local buckling in the corner regions of the cross-section, particularly in cold-formed sections at the location where the steel exhibits a reduced fracture strain. Cracks eventually form and gradually propagate through the cross-section under repeated cyclic loading. The initiation of local buckling and fracture is influenced by the width-to-thickness ratio as well as the applied loading history. There is also a strong dependence on brace slenderness, since for a given level of lateral deformation, higher curvature arise in plastic hinges that form in members with relatively low slenderness. This behaviour has prompted some researchers [8, 9] to suggest empirical relationships that evaluate ductility limits as a function of member slenderness. Clearly, both the cross-section and member slenderness have an influence on behaviour. It is therefore not surprising that fracture occurred in the 40x40x2.5 braces, due to a combination of relatively low member slenderness and high width-to-thickness ratio, coupled with significant ductility demand.

Whilst slender braces may perform better in terms of ductility capacity, as limited by local buckling and fracture, seismic codes usually impose an upper limit on $\bar{\lambda}$ which can typically be as low as 1.3 or 1.5. The purpose of this upper bound is normally to prevent elastic buckling and reduce the effects of pinched loops, low energy dissipation and sudden loading. However, the adoption of such low limits of $\bar{\lambda}$ can often become the controlling factor in the dimensioning of all frame members, leading to grossly uneconomic design. In the test series described in this paper, bracing members with $\bar{\lambda}$ of at least 2.0 exhibited generally satisfactory behaviour. Together with the potential increase in ductility capacity, as well as likely impact on design economy, this result supports some relaxation in the slenderness limits specified in design codes.

Regarding the performance of in-filled braces, the shake-table tests indicated that the behaviour was essentially similar to that of the steel counterparts, except for the larger brace sizes where the presence of the in-fill delayed local buckling and subsequently inhibited failure by section fracture. However, due to uncertainties related to the actual contribution of the infill to the inelastic cyclic behaviour as well as ductility, the use of composite members as dissipative diagonals in concentrically braced frames has not been addressed within Eurocode 8 provisions [10].

Further discussion of the main issues related to the performance of composite braces is presented elsewhere [17] as part of a complementary investigation which focused on the hysteretic response and ultimate behaviour of both hollow and in-filled members under quasi-static loading conditions. Under monotonic tensile loading the ductility of composite braces can be lower than that of steel members as expected. However, under cyclic loading, the delay in local buckling in more slender cross-sections can be a more decisive factor in providing higher ductility in composite members in comparison with steel counterparts. Evidently, on balance, the benefit obtained from the in-fill would only be realised in cases of relatively low member slenderness in which the local inelastic demand may be high, or in sections with comparatively high width-to-thickness ratios for which susceptibility to local buckling may be significant. These effects need to be appropriately assessed and quantified through more detailed analytical studies with a view to the provision of reliable guidance on design and detailing procedures.

CONCLUSION

Shake table tests performed on bracing members were briefly described, including the experimental arrangement, specimen configuration and loading procedures. Three sizes of square and rectangular hollow sections were employed to examine a range of member slenderness. Elastic tests were first performed to evaluate dynamic characteristics followed by inelastic tests to assess the response under earthquake loading. The inelastic tests provide essential data for validation of analytical and design studies, and offer immediate observations regarding the hysteretic dynamic response and failure criteria.

Based on experimental observations and assessment of fundamental behavioural aspects, a number of design considerations were discussed. These include issues related to different design approaches adopted for the treatment of brace behaviour in compression, and the implication of the idealisation involved on the lateral overstrength of the storey. Additionally, the direct influence of lateral overstrength on the ductility demand imposed on the bracing members was highlighted, indicating opposite trends in relationship with slenderness, depending on the design assumptions. In terms of available ductility, this was limited by brace failure through fracture of the cross-section at locally buckled regions. Although in-filling the braces inhibited this failure due to the delay in local buckling, the balance of this benefit depends on the width-to-thickness ratio as well as member slenderness. Overall, the results of this investigation also point out towards a need to reassess the slenderness limits imposed in some codes.

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