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On the dynamics of steel structures with X-type bracing

In steel construction X-type bracings are very often designed under the assumption that the compression diagonal will go slack and only the tension diagonal is accounted for in the structural analysis. With higher horizontal loads, when heavier diagonal sections are required, this assumption does not hold because the compression diagonal develops a considerable buckling load and thus will contribute to the horizontal stiffness of the bracing.
If we look at the dynamic behaviour of such a bracing we will find, that we cannot define a natural frequency in the classical sense, since the stiffness of the system is switching during its cyclic motion. This has also implications on the maximum amplitude which will develop under harmonic excitation. These effects are investigated with realistic cross section of a steel structure; conclusions for practical design are given.

Dynamisches Verhalten von Stahlbauten mit Kreuzverbänden. *Kreuzverbände in Stahlbauten werden häufig unter der Annahme bemessen, dass die Druckdiagonale schon bei kleinsten Lasten ausfällt. Verschiebungen und Eigenfrequenzen werden dann unter ausschließlichem Ansatz der Zugdiagonalen ermittelt. Dieser Ansatz kann für Diagonalen mit Rund- oder Flach-Querschnitten realitätsnahe sein. Wenn größere Horizontallasten abzutragen sind, wie dies häufig im Anlagenbau der Fall ist, werden z. B. Diagonalen aus Doppelwinkeln gewählt. Deren aufnehmbare Drucknormalkraft ist nicht mehr vernachlässigbar, so dass der oben beschriebene Ansatz nicht mehr realitätsnah ist.*
Im folgenden Artikel wird das dynamische Verhalten dieser Verbände genauer untersucht. Anhand eines stahlbautypischen Beispiels, welches einer Veröffentlichung von Çeltikçi et al. [1] entnommen ist, wird die tatsächliche Arbeitslinie eines Verbandsfeldes mit einem Schaltpunkt und unterschiedlichen inkrementellen Steifigkeiten entwickelt. Die Abschätzungen nach den üblichen ingenieurmäßigen Berechnungsverfahren werden einer numerischen Simulation gegenübergestellt. Als Ergebnis wird festgestellt, dass die vereinfachende Abschätzung, bei der nur die Zugdiagonalen angesetzt werden, hinsichtlich der dynamischen Amplituden auf der sicheren Seite liegt.

1 General

The economic design of steel structures in seismic areas requires conceptual decisions as to which type of bracing should be used. As pointed out by *Knoedel* and *Hrabowski* [2] it may not be economic to choose the highest possible behaviour factor in order to get the most reduction of the base shear: high effort may be required in design and manufacturing, and the structure might lose its robustness to structural changes, which are very often required in plant construction.

Fig. 1. Car park with X-type bracing after steel assembly
Bild 1. Parkdeck mit Kreuzverband im Bauzustand

For small and moderate peak ground accelerations (PGA) it seems to be sensible to go into ductility class low and cope with increased base shear. This is easy to handle without additional effort in design and only little additional mass is needed for the members of the bracing structure.

When designing with response spectra it may be beneficial to evaluate the structure's natural frequency if the vibration period is outside the constant spectral acceleration branch. Having a closer look at steel structures with X-type bracing systems (fig. 1) we will find that the natural frequency is not easy to define, since the stiffness of the system is altered while looping through a vibration cycle.

2 Terms, definitions and assumptions

We consider a five-storey bracing tower as given by *Çeltikçi* et al. [1]: 4 m storey height and 6 m span, having HEA 320 columns and IPE 450 walers. Depending on the mechanical model used the diagonals vary from 2 L 65×7 to 2 L 150×15. Inclination of the diagonals against horizontal is $\alpha = \arctan\left(\frac{4}{6}\right) = 33{,}7°$.

The characteristic wind loads per storey are 65 kN + 4 × 130 kN, summing up to a total characteristic base shear of 585 kN. The design base shear amounts to 878 kN, where horizontal loads from sway-imperfections are neglected for simplicity.

While we adopt the example so far we make additional simplifications and assumptions:

- All storeys are braced with X-type diagonals 2 L 100 × 10.
- A total mass of 40 metric tons is associated to each column from the adjacent building, which is a mean of 8 tons per column and storey. Further on it is assumed that the bracing tower is to stabilise a total of 6 columns (1/2 + 5 + 1/2), corresponding to a building width of 36 metres. Thus the total mass to be stabilised by the bottom storey bracing system is 240 metric tons.
- A seismic peak ground acceleration a_g of 2m/s^2, a soil factor S of 1,0 and a behaviour factor q of 1,5 are assumed, resulting in a maximum design base shear (constant spectral acceleration branch according to EC8 section 3.2.2.5) of

$$V_{\text{base shear}} = M \cdot a_g \cdot S \cdot \frac{2{,}5}{q} \qquad \text{(eq. 1)}$$

$$V_{\text{base shear}} = 240000 \cdot 2{,}0 \cdot 1{,}0 \cdot \frac{2{,}5}{1{,}5} = 240000 \cdot 3{,}33 = 800\,\text{kN}$$

- Two different quantities are used to characterise the natural frequency (eigenfrequency) of the system: angular frequency ω [radians per second] or (temporal) frequency f [cycles per second = Hz], where $\omega = 2\,\pi\,f$.
- Two different quantities are used to characterise damping of the system: the percentage of critical damping D, often referred to as ζ (zeta). In EC8 [3] this quantity is referred to as ξ (xi). The other quantity is the logarithmic decay δ, which is given by $\delta = 2\pi\,D$.
- The members of the bracing have initial global imperfections. Under increasing longitudinal compression they deviate monotonously rather than flip aside when *Euler*'s buckling load is approached.
- With multi-storey bracing towers it will be assumed that only the cantilever mode will be governing. If we do not look for the columns' compressive forces it is sufficient to look at the motion of the lowest storey and to put the total masses of the adjacent building on the top ends of the columns.
- The bracing tower is assumed to be a truss-type structure. Unintended joint rigidity and unintended clamping of the columns' bases, which might add considerable stiffness to the structure [4] are not considered.

3 Design for static loading

When designing an X-type bracing for a steel structure we usually assume that both diagonals have no compressive strength. Depending on the direction of the horizontal load which would cause a storey drift, only one out of the pair of diagonals would be activated in tension, the other one is assumed to fail by buckling under zero load. This is a reasonable assumption for rods, flats or small L-shaped sections which are very often used for bracing diagonals. Compared to single bracing diagonals, which are loaded alternatively in tension and compression, this method allows to have very light bracings, because those members' tensile strength is independent of their lengths. It is also very effective for the structural engineer, because a simple net cross section check is sufficient to design the tensile member after having determined the required strength by use of

$$N_{\text{diagonal}} = \frac{V_{\text{base shear}}}{\cos\alpha} \qquad \text{(eq. 2)}$$

However, the assumption that the compression members would fail at very small loads does not hold for typical bracing towers with considerably high horizontal loads, which require e. g. double L 100 × 10 sections for a diagonal. As we know from global buckling of a beam according to second order theory, we can expect the resistance of a buckled bar being close to the *Euler*'s load, as long as we stay with moderate deflections. *Çeltikçi* et al. [1] included this effect in their design procedure and added the buckling load of the compression diagonal to the tensile resistance of the tension diagonal, which leads to smaller cross sections needed for the bracing diagonals.

4 Approaching natural frequency

When determining the natural frequency of the sway mode of the structure, the same assumption as described above is usually made for the static loading design: the compression diagonal is assumed to fail at zero load, which means that only one of the diagonals at a time is contributing to the horizontal stiffness of the storey. Therefore only one diagonal is used when determining the natural frequency e. g. by means of a *Rayleigh-Morleigh* procedure [5].

Following the principle of virtual work we can denote the horizontal head displacement of a n-storey bracing tower with equal horizontal loads F per storey. In the case of a *Rayleigh-Morleigh* procedure the self weight of all oscillating masses is imposed in horizontal direction (fig. 2). For one tension diagonal per storey we receive the following contributions of the different members to the total displacement:

in general $$w = \sum \frac{N \cdot N}{E \cdot A} \cdot L \qquad \text{(eq. 3)}$$

luff column $$w = \frac{F \cdot H}{E \cdot A_C} \cdot \frac{H^2}{B^2} \cdot \sum_{i=1}^{n-1} (+i)^2 \qquad \text{(eq. 4)}$$

lee column $$w = \frac{F \cdot H}{E \cdot A_C} \cdot \frac{H^2}{B^2} \cdot \sum_{i=1}^{n} (-i)^2 \qquad \text{(eq. 5)}$$

diagonals $$w = \frac{F \cdot B}{E \cdot A_D} \cdot \frac{1}{\cos^3\alpha} \cdot \sum_{i=1}^{n} (+1 \cdot i) \qquad \text{(eq. 6)}$$

walers $$w = \frac{F \cdot B}{E \cdot A_W} \cdot \sum_{i=1}^{n} \left(i - \frac{1}{2}\right) \qquad \text{(eq. 7)}$$

Remark: The given expression for the contribution of the waler contains full compression of the top waler, i. e. the displacement of the luff column's head is described. For the displacement of the middle of the top waler the above sum must be reduced by 1/4.

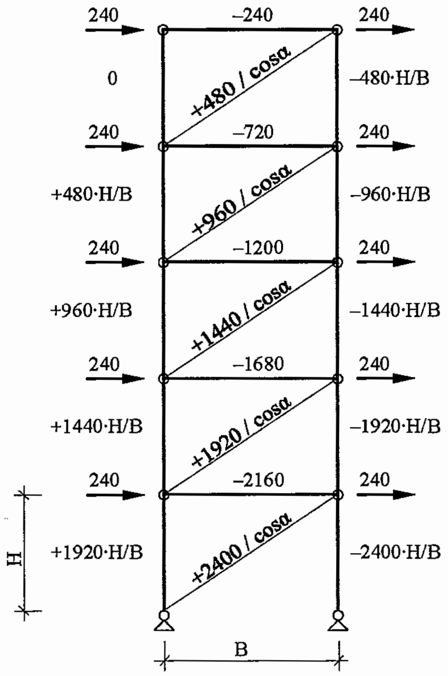

Fig. 2. 5-storey bracing tower with 480 kN horizontal force per storey from self weight – truss member forces in kN
Bild 2. 5-geschossiger Verbandsturm mit 480 kN Horizontallast je Stockwerk aus Eigenlasten – Stabkräfte in kN im Fachwerk

Evaluating with the values given above we receive:

luff column $$w = \frac{480 \cdot 4}{2{,}1 \cdot 10^4 \cdot 124} \cdot \frac{4^2}{6^2} \cdot 30 = 10 \text{ mm} \quad \text{(eq. 8)}$$

lee column $$w = \frac{480 \cdot 4}{2{,}1 \cdot 10^4 \cdot 124} \cdot \frac{4^2}{6^2} \cdot 55 = 18 \text{ mm} \quad \text{(eq. 9)}$$

diagonals $$w = \frac{480 \cdot 6}{2{,}1 \cdot 10^4 \cdot 38{,}4} \cdot \frac{1}{(\cos 33{,}7°)^3} \cdot 15 = 93 \text{ mm} \quad \text{(eq. 10)}$$

walers $$w = \frac{480 \cdot 6}{2{,}1 \cdot 10^4 \cdot 98{,}8} \cdot \frac{25}{2} = 17 \text{ mm} \quad \text{(eq. 11)}$$

The total horizontal head displacement of the bracing tower is

$$w = 10 + 18 + 93 + 17 = 138 \text{ mm} \quad \text{(eq. 12)}$$

The corresponding lowest natural frequency according to *Rayleigh-Morleigh* is

$$f = \frac{1}{2 \cdot \pi} \cdot \sqrt{\frac{g}{0{,}138}} = 1{,}34 \text{ Hz} \quad \text{(eq. 13)}$$

with a vibration period of $T = 0{,}75$s.

This frequency is a lower bound because in a truss model there are some unintended stiffnesses apart from the finite buckling stiffness of the compression diagonal, which will be described below. There are constructional clampings at the waler-column connections and at the diagonal-gusset connections, and the columns are continuous through the storeys, all three of which put additional stiffness to the system. Furthermore there are constructional clampings at the bottom ends of the columns, which may reduce the buckling length of the bottom section of the columns considerably [4]. Therefore in critical cases we increase the eigenfrequency arbitrarily by 10 % which corresponds to an increase of overall-stiffness of app. 20 %.

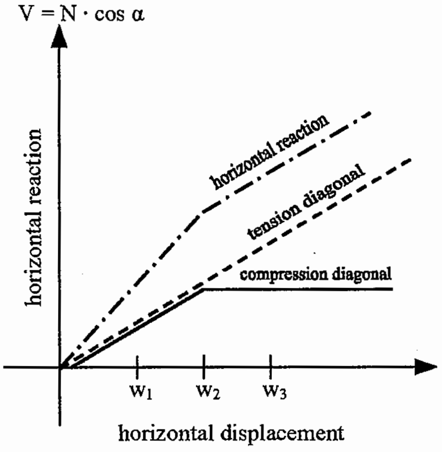

Fig. 3. Load-displacement-curves of the diagonals and total restoring force
Bild 3. Last-Verschiebungs-Kurven der Diagonalen und Gesamt-Rückstellkraft

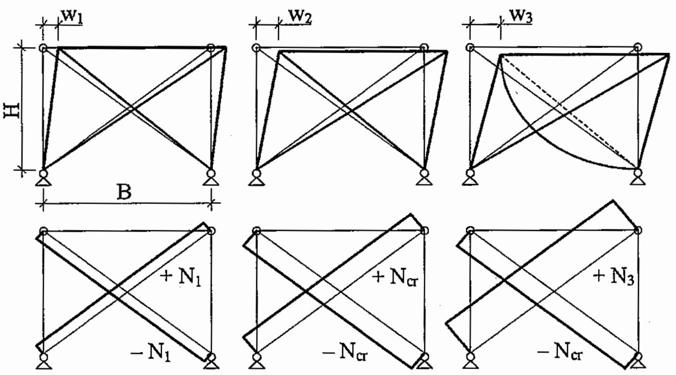

Fig. 4. Storey drift and bracing forces for a compression diagonal with finite stiffness
Bild 4. Stockwerksverschiebung und Verbandskräfte für Druckdiagonalen mit endlicher Steifigkeit

If we make use of the assumption that the compression diagonal has a finite buckling load, we see that we have a switching incremental stiffness in the bracing system (fig. 3).

- For a storey drift which causes compressive forces below *Euler*'s load we have full stiffness of both diagonals present (displacement w_1 in fig. 4).
- If the storey drift passes this limit (displacement w_2 in fig. 4) the restoring force of the compression diagonal remains constant while the restoring force of the tension diagonal increases proportional to the storey drift (displacement w_3 in fig. 4). In terms of incremental stiffness our initial stiffness is reduced by a factor of 2 if both diagonals are built with the same cross sections (fig. 3).

Using the classical definition of eigenfrequency for a single DOF spring-mass-oscillator

$$\omega = \sqrt{\frac{c}{m}} \quad \text{(eq. 14)}$$

we see that the natural frequency is altered by a factor of $\sqrt{2}$, depending on which of both stiffnesses we have in the system.

5 Non-harmonic periodic motion

For showing the behaviour of a dynamic system with switching stiffness (see [6]) only the bottom storey of the bracing tower is used for simplicity. The horizontal displacement of the bottom storey with one diagonal 2 L 100 × 10 under a horizontal load of 2 × 1200 kN is (evaluated as given above)

$$w = 0 + 1{,}6 + 31 + 3{,}5 = 36{,}1 \text{ mm} \qquad \text{(eq. 15)}$$

For the same situation with X-type diagonals 2 L 100 × 10 we receive

$$w = 0 + 1{,}6 + 31/2 + 3{,}5 = 20{,}6 \text{ mm} \qquad \text{(eq. 16)}$$

The natural frequencies are 2,62 Hz with T = 0,38 s for the former and 3,47 Hz with T = 0,29 s for the latter. The horizontal storey drifting stiffnesses are

$$c = \frac{2400}{36{,}1} = 66{,}5 \frac{\text{kN}}{\text{mm}} = 66500 \frac{\text{kN}}{\text{m}} = 66{,}5 \cdot 10^6 \frac{\text{N}}{\text{m}} \qquad \text{(eq. 17)}$$

with the presence of one diagonal and

$$c = \frac{2400}{20{,}6} = 116{,}5 \frac{\text{kN}}{\text{mm}} = 116500 \frac{\text{kN}}{\text{m}} = 116{,}5 \cdot 10^6 \frac{\text{N}}{\text{m}} \qquad \text{(eq. 18)}$$

with X-diagonals.

If we engage two different stiffnesses c_i in the mass-spring-oscillator we should expect that the duration of the vibration period T is bounded by

$$T_i = \frac{2 \cdot \pi}{\omega_i} = 2 \cdot \pi \cdot \sqrt{\frac{m}{c_i}} \qquad \text{(eq. 19)}$$

with i = 1,2

The motion is non-harmonic because the displacement's time history is altered between both

$$w_i(t) = A \cdot \sin(\omega_i \cdot t) = A \cdot \sin\left(t \cdot \sqrt{\frac{c_i}{m}}\right) \qquad \text{(eq. 20)}$$

within the same half-wave of a cycle.

In order to simulate this numerically we provide further information on the compression diagonal at this point. The global buckling load of a diagonal 2 L 100 × 10 – S235 with full length of 7211 mm amounts to

$$N_{b.R.d} = 0{,}130 \cdot 820 = 106 \text{ kN} \qquad \text{(eq. 21)}$$

which corresponds to a horizontal storey load (base shear) of

$$N_{b.R.d} = 106 \cdot \cos(33{,}7°) = 88 \text{ kN} \qquad \text{(eq. 22)}$$

This corresponds to a storey drift of

$$w_{cr} = \frac{88}{66{,}5} = 1{,}32 \text{ mm} = w_2 \qquad \text{(eq. 23)}$$

where the stiffness in the system will alter according to fig. 3.

If the diagonals are interconnected at their crossover and we assume the buckling length to be 3605 mm we have a buckling load of 331 kN, corresponding to a base shear of 275 kN and a critical drift of 4,14 mm.

According to the parameters chosen we subject the system to a harmonic excitation with a prescribed force amplitude of

$$\hat{F} = 240000 \cdot 2{,}0 \cdot 1{,}0 = 480 \text{ kN} \qquad \text{(eq. 24)}$$

and a period which is identical to the natural frequency of the system. We compare a damping of δ = 0,015 or D = 0,24 %, which might be the lower bound for steel structures, and $\delta = \pi/10$ or D = 5 % which is assumed in EC8 [3] for buildings, when design response spectra are used. The time-history-plots are given in fig. 5, the corresponding phase-plots are given in fig. 6.

The limit amplitude of a forced motion with small damping ratio is expected to be

$$\hat{w} = w_{dynamic} = w_{static} \cdot \frac{\pi}{\delta} = \frac{w_{static}}{2D} \qquad \text{(eq. 25)}$$

$$\hat{w} = w_{dynamic} = \frac{480}{116{,}5 \cdot 2 \cdot 0{,}0024} = 858 \text{ mm} \qquad \text{(eq. 26)}$$

$$\hat{w} = w_{dynamic} = \frac{480}{116{,}5 \cdot 2 \cdot 0{,}05} = 41 \text{ mm} \qquad \text{(eq. 27)}$$

These values are confirmed by a numerical simulation with the same model, but are not presented here due to restrictions of space. It should be noted, that with the larger damping ratio steady state is obtained after 10 to 15 cycles, whereas with the smaller damping ratio some 150 cycles are needed to attain 90 % of the steady state amplitude.

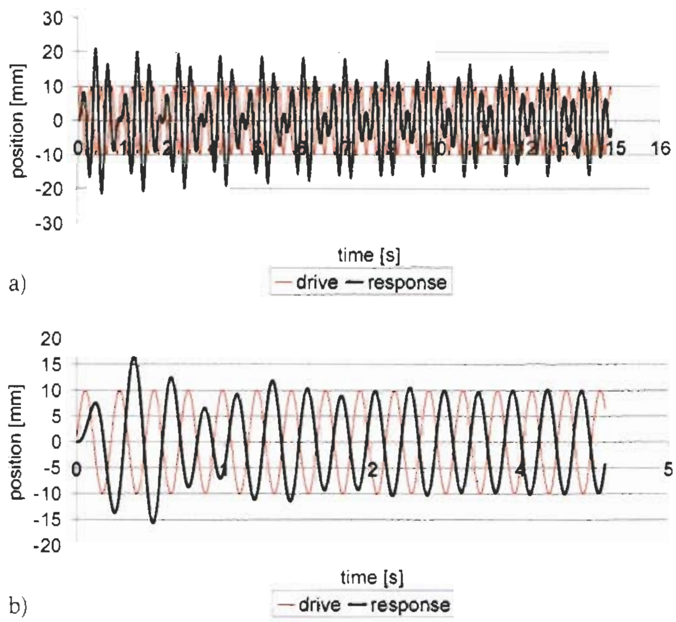

Fig. 5. Time-history-plots; diagonals are not connected at their crossover; a) D = 0,24 %; b) D = 5 %
Bild 5. Zeitverläufe; die Diagonalen sind am Kreuzungspunkt nicht verbunden; a) D = 0,24 %; b) D = 5 %

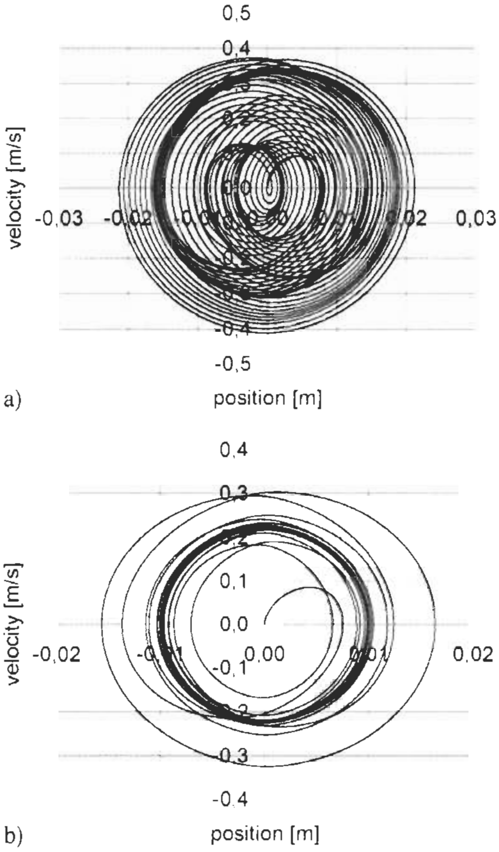

Fig. 6. Phase-plots showing limit cycles; diagonals are not connected at their crossover; a) D = 0,24 %; b) D = 5 %
Bild 6. Phasen-Portraits mit Grenzzyklen; die Diagonalen sind am Kreuzungspunkt nicht verbunden; a) D = 0,24 %; b) D = 5 %

Both time-history-plots show (fig. 5) that the amplitudes of the system are reduced considerably through the switching stiffness mechanism. This effect is known [7], it can be used to reduce vibration amplitudes without dissipation. The vibration periods are adapting to the period of the forced motion. As known to the author the question of a critical driving frequency for systems with switching stiffness has not been answered so far. In fact this answer does not seem to be important for practical design purposes, since such systems are usually simulated under recorded or virtual acceleration spectra, where the question of a critical driving frequency is not important.

In fact the oscillating periods have been tested in a dying out simulation. With zero-damping there is no decay and the vibrations can be evaluated easily. The periods observed differed by a factor very close to $\sqrt{2}$, depending on the position of the switch. If the switching down of the stiffnesses takes place very close to the maximum displacement (or virtually beyond) then most (or all) of the cycle is governed by the bigger of both stiffnesses, the period will be shortest. If the switching point lies at a very small amplitude (or at zero) then most (or all) of the cycle is governed by the smaller of both stiffnesses, the period will be longest.

According to EC8 [3] (load see above) and the stiffnesses obtained above we should expect a steady state bottom storey drift between

$$w = \frac{800}{66{,}5} = 12 \text{ mm} \qquad \text{(eq. 28)}$$

if only one diagonal is activated and

$$w = \frac{800}{116{,}5} = 7 \text{ mm} \qquad \text{(eq. 29)}$$

if two diagonals are activated.

We can see in fig. 5, that the time-history-plots show a steady state motion within these bounds.

6 Further considerations

- If the driving frequency is close to the bending frequency of the members – or close to the string-type frequency, if the bracing diagonals are prestressed – parametric excitation may occur in a way that the system develops negative damping. In this case the bracing does not restrain the structure any longer (for a more detailed explanation see e. g. *Nölle* [8]). This effect is not considered in the present paper.
- If plastic design is involved, e. g. by using ductility class medium or high in seismic design (DCM or DCH according to EC8 [3]), the change in incremental stiffness is even more pronounced. In this case the behaviour will become highly non-linear (*Knoedel* and *Hrabowski* [2], see also *Vayas* [9]).
- If the above 5-storey bracing tower was analysed like the bottom storey as shown above, the diagonals in the different storeys would buckle at different times, which would increase the non-linear effects.

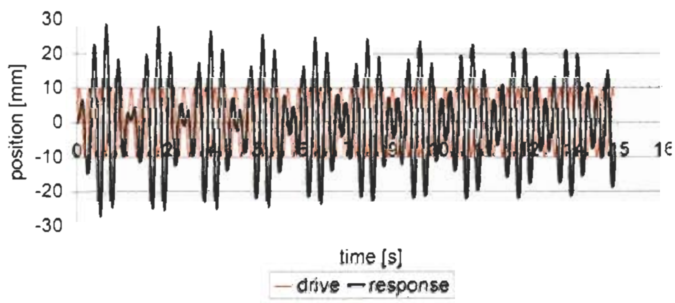

Fig. 7. Time-history-plot; diagonals are interconnected at their crossover; D = 0,24 %
Bild 7. Zeitverlauf; die Diagonalen sind am Kreuzungspunkt verbunden; D = 0,24 %

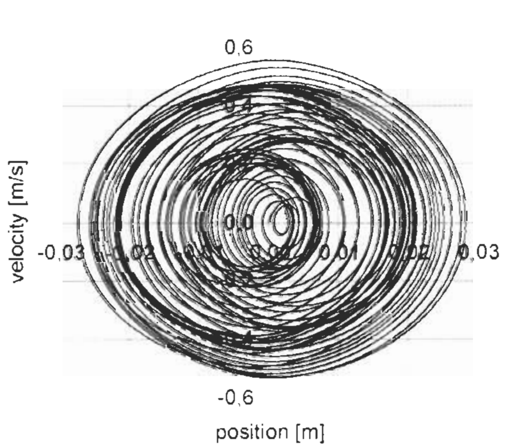

Fig. 8. Phase-plot showing limit cycle; diagonals are interconnected at their crossover; D = 0,24 %
Bild 8. Phasen-Portrait mit Grenzzyklus; die Diagonalen sind am Kreuzungspunkt verbunden; D = 0,24 %

- If the diagonals of the X-type bracing are interconnected at their crossover this will lead to larger response amplitudes. With the above parameters the maximum amplitude will increase from 21 mm (see fig. 5 and fig. 6 for D = 0,24 %) to 28 mm (see fig. 7 and fig. 8 for D = 0,24 %). This is due to the higher buckling load of the single diagonal sections which causes an increasing critical storey drift. Therefore the system remains longer in the linear state and is able to absorb more of the offered energy.

7 Conclusions

For practical design purposes we can conclude:

- Evaluating the classical eigenfrequency for one or two active diagonals gives bounds for the actual frequency of the non-harmonic periodic motion of the system.
- It is sufficient to estimate the lowest natural frequency of the system by taking only the tension diagonals into account. The calculatory displacements will be larger than those of the actual system.
- Due to the effective damping the system's response will always be more benign than expected by use of the damping values widely accepted for steel structures, such as D = 0,24 %. Note that this does not hold for system identification, when the task is to have a representation of the dynamic system, which is as close to reality as possible.
- From a seismic design point of view it is unfavourable to interconnect the crossing diagonals. However this might be different with plastic design.

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Dissertationen

Untersuchungen zum Material- und Tragverhalten von Schrauben der Festigkeitsklasse 10.9 während und nach einem Brand

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(TU Darmstadt)

Die brandschutztechnische Bemessung hat sich in den letzten Jahrzehnten grundlegend gewandelt. Während die Brandschutznachweise in der Vergangenheit fast ausschließlich auf der Grundlage von Bemessungstafeln geführt wurden, bieten die heutigen Eurocodes (Teile 1-2) ein erweitertes Spektrum an Nachweisformaten an. Die Bemessungskonzepte der Eurocodes basieren auf den temperaturabhängigen Materialeigenschaften des jeweils betrachteten Bauteils. Das Werkstoffverhalten marktüblicher Baustähle ist heute weitestgehend erforscht und die Ergebnisse liegen in Form von temperaturabhängigen Spannungs-Dehnungs-beziehungen im Eurocode vor. Das Materialverhalten von Schrauben der Festigkeitsklasse 10.9 ist im Gegensatz hierzu unbekannt. Die Schraubentraglast im Brandfall wird nach dem Eurocode in Abhängigkeit von temperaturabhängigen Reduktionsfaktoren berechnet.

Auf Grund des geringen Kenntnisstandes auf diesem Gebiet wurden in der vorliegenden Arbeit Versuche an Zugproben und Schraubengarnituren durchgeführt, um das Material- und Tragverhalten hochfester Schraubengarnituren im Brandfall zu untersuchen. Die Zugversuchsergebnisse wurden im Anschluss analytisch formuliert und für numerische Simulationen von Schraubengarnituren verwendet. Die Versuchs- und Berechnungsergebnisse ermöglichen zum einen die Verifikation der normativ festgelegten Reduktionsfaktoren und zum anderen ein besseres Verständnis für das Tragverhalten hochfester Schraubengarnituren im Hochtemperaturbereich.

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