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Behavior of Steel Frames Subjected to Repeated and Reversed Loads

Comportement des portiques multi-étagées en acier sous l'effet de charges répétitives et alternatives

Das Verhalten von Stahlrahmentragwerken unter Einfluß periodisch veränderlicher Wechsellasten

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1) Introduction

Recent work in earthquake engineering has centered around full scale dynamic testing of multi-story buildings (Refs. 1 and 2) and computer studies of the behavior of simple systems under recorded earthquake motions or models thereof (Refs. 3, 4 and 5). Some tests have been performed to study the behavior of steel and concrete beams and frames under simulated wind, earthquake or impact loads. In recent tests at the University of California at Berkeley cantilever beams were tested under cyclic loads to study the behavior of these beams near the connecting zone (Refs. 6 and 7). In addition, as adjuncts to recent tests of multi-story frames at Lehigh University to study the static behavior under a monotonic load application, four frames were tested under a reversed loading after large inelastic deformations had occurred (Refs. 8 and 9). Currently available methods of analysis were found to adequately describe the static behavior of these test frames under the combined effect of gravity and monotonically increasing lateral loads. However, these methods were found to be inadequate to describe the static behavior of the frames under reversed loading even for relatively simple structures.

A research program has been initiated at Lehigh University in order to extend plastic design concepts to the design and analysis of structures subjected to seismic loadings. In the experimental portion of this program, two series of tests on single and multi-story frames were planned. This discussion gives a brief account of the first series of tests which has been completed recently.

2) Design of Test Frames

The test frames involved in the first series were designed to be typical of current aseismic design practice. The prototype frame was an eight-story, single-bay structure. A bay width of 15 feet, story height of 10 feet and bent spacing of 18 feet were selected for the prototype frame shown in Fig. 1.

A three-story assemblage was designed to represent levels 5, 6 and 7 from the top of the building from which a single story frame representing level 7 was selected for the initial test. Half-story columns above level 5 and below level 7 were used to locate the point of inflection in the double curvature columns. The two frames in the first series are shown in Fig. 1 in their relative position with respect to the prototype frame.

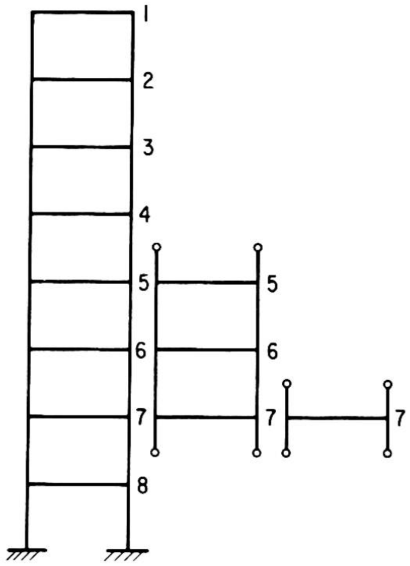

Fig. 1 Prototype Frame and Test Frames

The design and therefore the testing of the frame utilized a single horizontal load applied to the top of the assemblage. The frame was designed for constant story shear because for an eight-story frame the variation in the total aseismic design shear (Ref. 10) in the lower stories is usually small (Ref. 11). In addition, the envelope of maximum dynamic shear obtained from several modes of a shear type building has small variations in the lower portions of such a building (Ref. 12).

The gravity loadings used in the design were 80 psf full live load and 80 psf dead load on all the floors. An average live load reduction of 40% was used for both beams and columns. The working horizontal load was the summation of the design shears from the top of the structure down to and including the component at level 5. The working shear was equal to about 3½ percent of the sum of the dead loads through level 7.

The design also incorporated a ratio of column stiffness to beam stiffness which was selected to be representative of buildings designed using current aseismic design practice in California.

The plastic design method which was used to determine the members initially assumes no P-Δ effect and a likely-to-occur mechanism (Ref. 13). A plastic moment balancing analysis then was used to check that all moments are less than or equal to their fully plastic values. From the resulting moment diagram and sections required, the Δ's were calculated and the P-Δ moments were found. Redesign then included this P-Δ effect and the sections required initially were altered when necessary.

Once the above set of members were selected, the frame was analyzed using the computer program (Ref. 14) described in the next section. By using this program repeatedly the members were selected such that they satisfied the requirements of aseismic design practice.

In summary, the three-story frame was designed and analyzed plastically and then checked by the allowable-stress method. The single-story frame was selected as a duplicate of the lower floor of this frame. The resulting member sizes selected were an 8WF40 section for the columns and a 10WF29 section for the beams. The member sizes and frame geometry for both frames are shown in Fig. 2.

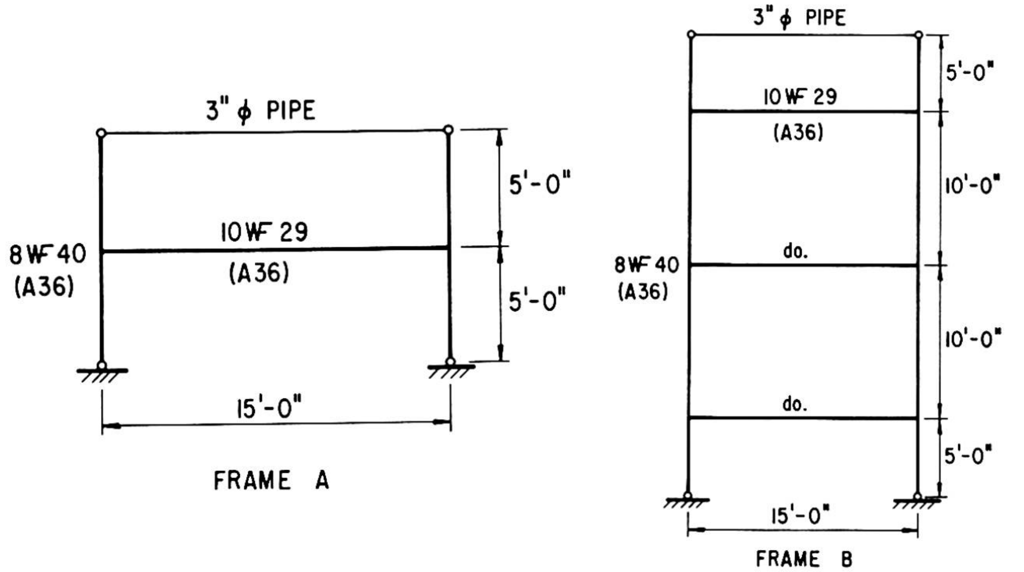

Fig. 2 Geometry and Member Sizes for Test Frames

3) Analyses of Test Frames

When the frames were analyzed under the combined earthquake and gravity loads, the change in member stiffness due to axial force, the overturning effects of the lateral load and the P-Δ moment were included.

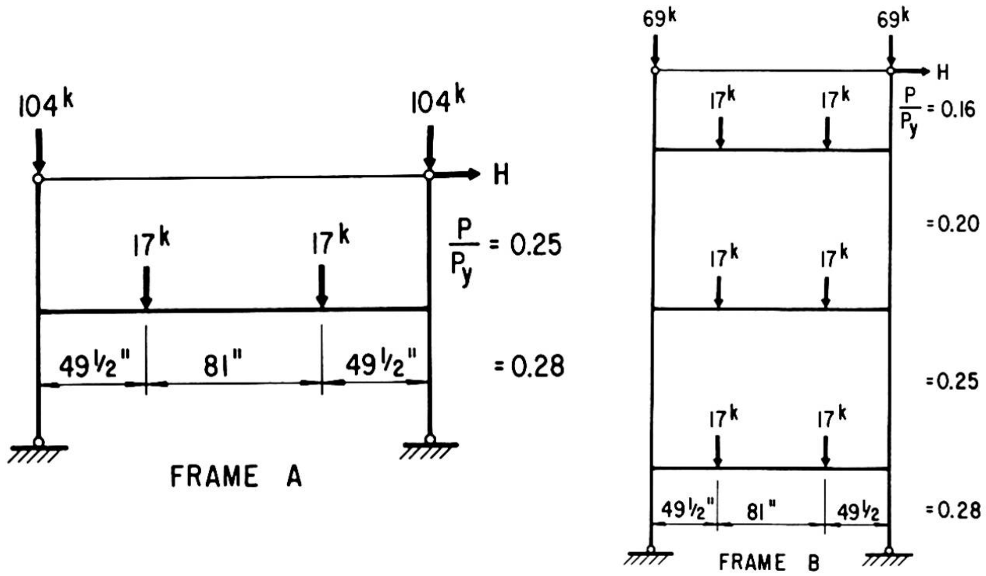

Fig. 3 Loads and Axial Load Ratios for Test Frames

At the working level of the monotonically applied horizontal load and the gravity loads shown in Fig. 3 the results of this second-order analysis were used to check the adequacy of the beams and columns with the AISC interaction formulas and satisfactory results were obtained. (In addition, the members of both frames were checked under the working level of gravity load only).

The analysis of each frame was then continued into the inelastic range past the point of frame instability. The load-deflection curves for both frames were essentially the same as shown in Figs. 4a and 4b.

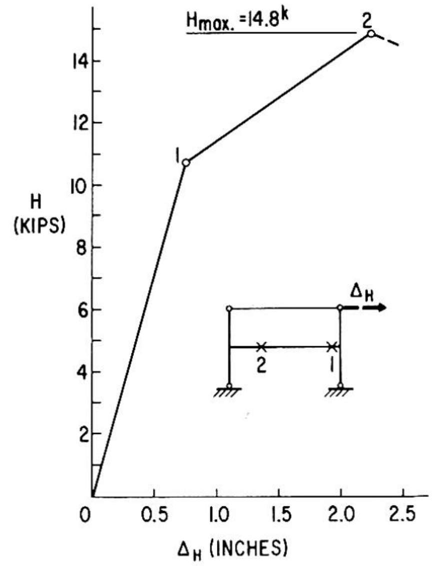

Fig. 4a Load-Deflection Curve for Frame A

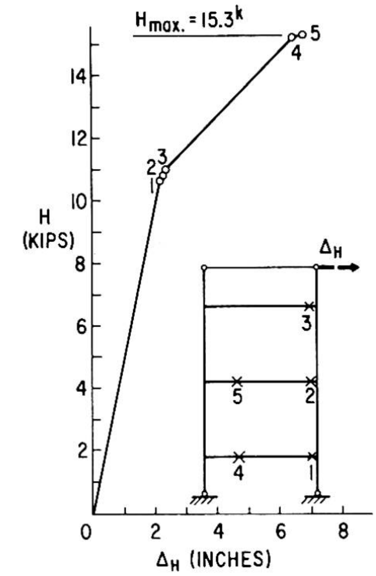

Fig. 4b Load Deflection Curve for Frame B

For the single-story frame the frame instability load and mechanism load coincide at a lateral load of 14.8 kips and at a deflection of 2.26 inches at the point of load application. However, the three-story frame became unstable at a load of 15.3 kips and a corresponding deflection of 6.83 inches before formation of a failure mechanism.

The single-story frame had a combined mechanism at its maximum load with the first hinge forming at the leeward end of the beam and the second hinge at the windward load point on the beam. The three-story frame had a similar pattern of hinge formation with the first hinge forming at a load of 10.7 kips in comparion to the working value of 5.2 kips. Since the ratio of maximum load to the working load is 2.9, a considerable savings could have been realized by utilizing more of the inelastic strength of the frame in design while keeping within acceptable drift limitations. In fact, a 13% lighter frame using 8W35 columns and 10W25 beams was analyzed under factored gravity plus lateral loads (L.F. = 1.30) (Ref. 13). The maximum load in this case was 8 kips which is considerably higher than the factored lateral load of 6.75 kips. However, for the former three-story frame which was designed for a lateral load of 3½ percent of total dead load, the ultimate value of lateral load is about 10 percent of the working dead load.

The above analyses were based on handbook values for cross-sectional properties and on the nominal static yield stress of 36 ksi specified for ASTM A-36 steel. The analyses were repeated after the cross-sectional shapes of the actual members used in the frames were measured and after the static yield stress levels were determined by testing tension specimens cut from adjacent pieces of the same length of steel. All material used was gag straightened by the producer.

4) Test Setup and Loading Program

The two frames were tested under constant (working) gravity loads and a program of statically applied cyclic horizontal displacements of the top of the frames similar to those used by E. P. Popov on the cantilever beams (Refs. 6 and 7).

Two unique devices were used to load and to brace the frames without offering any restraint to in-plane movements. Gravity-load simulators were used to apply the constant vertical loads to the quarter points of the beams through a spreader beam and to the column tops, and bracing linkages were used to prevent out-of-plane movements of the members of the frames (Ref. 15). The horizontal displacement was produced by mechanically displacing the top of the frame. Overall views of the test setups for the two frames are shown in Fig. 5.

Zero-moment end conditions were imposed on the ends of the columns at the assumed points of inflection above and below the main portions of each frame. Pinned-bases utilizing roller bearings were used at the lower end of each of the lower half-story columns. A pinned-end tie beam between the two ends of the top half-story columns was used. to distribute the horizontal force.

Displacements and rotations of various points throughout the frame were measured mechanically and electrically. Strain gages were used extensively throughout the structure. Computations from the strain gage readings and the measured deflections of the gaged points reduce the frames to determinate components.

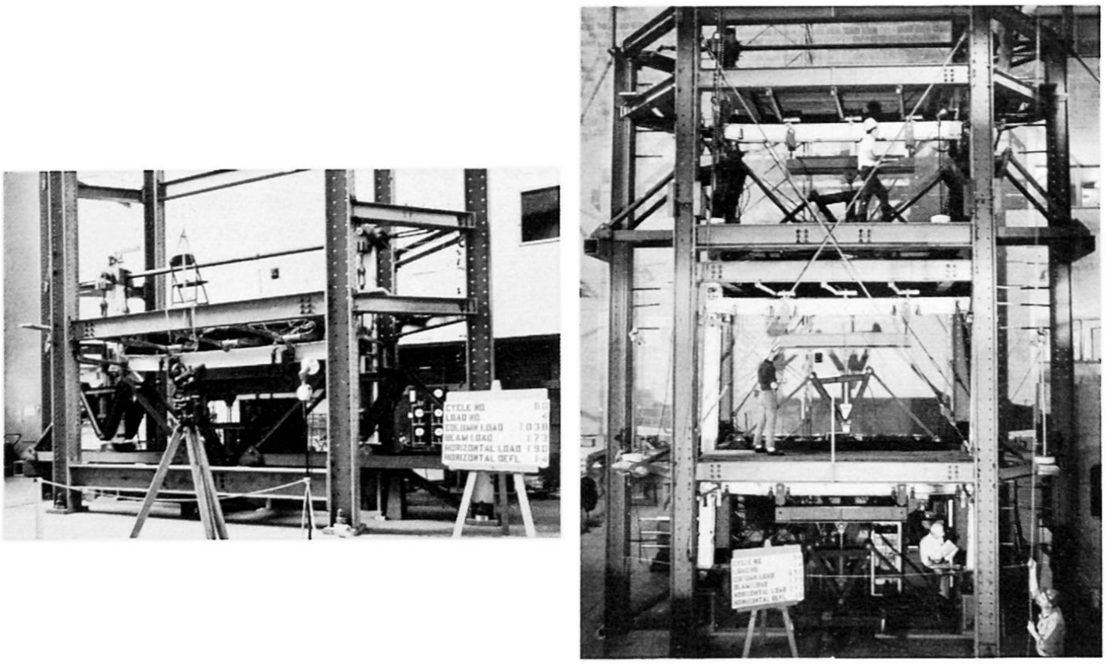

Fig. 5 Test Arrangements for Frame A and Frame B

Initially the gravity loads were applied to the frames and then sets of lateral displacements of increasing amplitudes were applied to the frames in a cyclic manner. In each case, the amplitudes to be cycled were selected to bracket the plastic hinge occurrences and other intermediate points on the respective load-deflection curves. For displacements in the elastic range three cycles were used at each amplitude and for inelastic range displacements five cycles were used. The number of replications at each amplitude was set to observe the stability of the hysteresis loops at the various amplitudes of deflection and inelastic conditions of the frames. The amplitudes selected for Frame A are superimposed on the load-deflection curve as shown in Fig. 6. The resulting displacements program is also given.

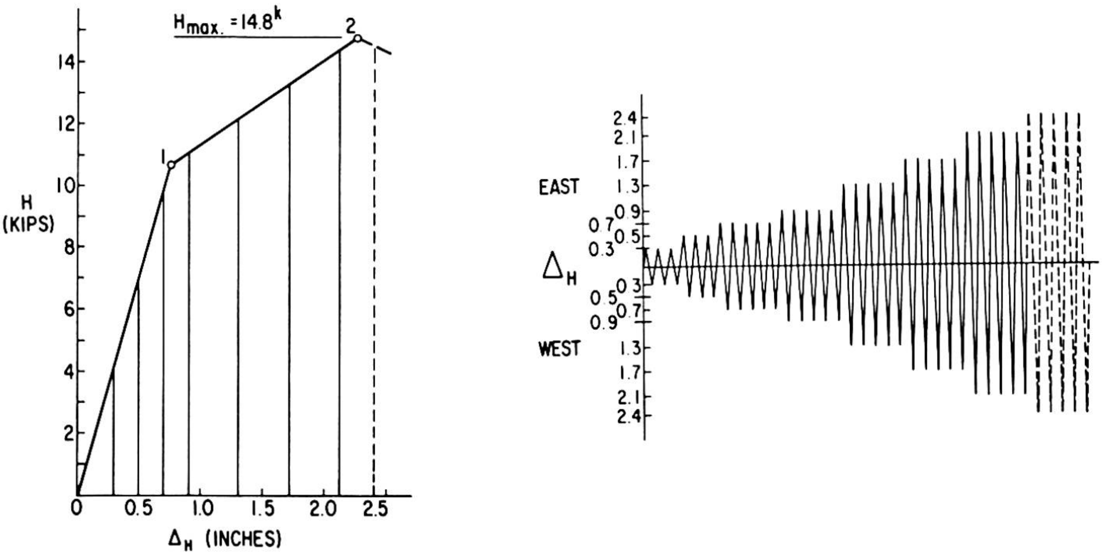

Fig. 6 Cycling Amplitudes and Horizontal Displacement Programs for Fram

During the tests, complete sets of static readings were taken at suitable intervals to permit construction of the hysteresis loops.

5) Test Results

Sixty cycles of horizontal displacement with increasing amplitudes were applied to the single-story frame with a maximum displacement of 5.2 inches which is 14 times the deflection at working load and 2.3 times the deflection at the maximum horizontal load. The three-story frame had 54 cycles at various amplitudes of displacement applied to it with a maximum cycled displacement of 10 inches. (At the end of the test 13.5 inches were applied in one direction). The displacement is 9 times the working load displacement and 1.5 times the deflection at the maximum predicted load. The ratios given above indicate the toughness and ductility of these steel frames. Cycles at selected amplitudes are shown in Fig. 7 for Frame B.

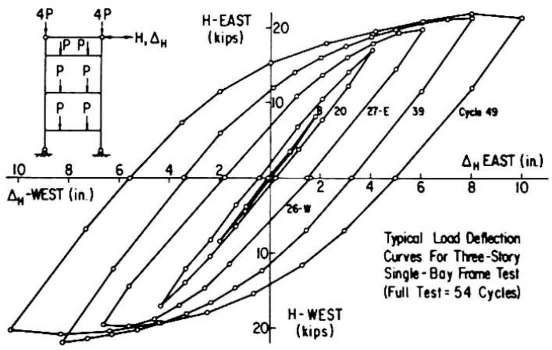

Fig. 7 Selected Load-Deflection Curves for Frame B

For the single-story frame the deflection at which the maximum load was reached was predicted closely by the monotonic analysis. But, for the three-story frame the maximum load occurred at a somewhat higher deflection (about 8 inches compared to the 6.8 inches predicted).

In addition, the replications of cycles at all amplitudes, even those beyond the frame instability deflection, showed stable results. This stability of the loops is indicated in Fig. 8 for the largest cycled amplitudes during the test of Frame A.

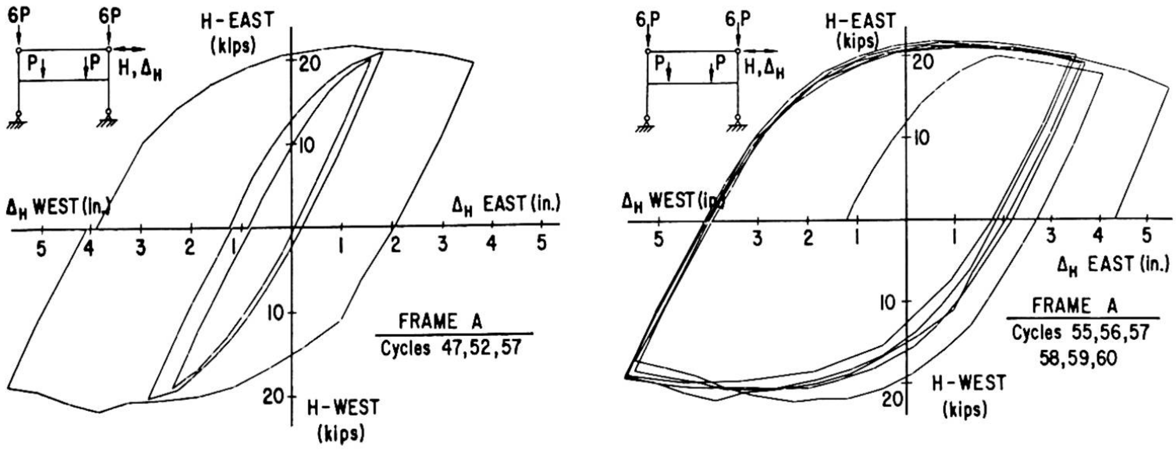

Fig. 8 Selected Load-Deflection Curves and Stability of Load-Deflection Curves for Frame A

Both tests showed a considerable reserve capacity for steel frames when subjected to cyclic lateral displacements. In each case, the maximum load the frame could withstand was about 40 percent greater than that predicted by the second-order elastic-plastic analysis of the frames under monotonically increasing lateral loads. (This percentage was computed after the analysis was redone with the actual experimental plastic moment values.)

One significant factor which tends to increase the lateral load over that predicted previously is the actual location of the plastic hinges in the beams. The analysis assumes no finite size for the beam-to-column connection whereas the yielding for the initial hinges was centered about one-half the depth of the beam from the column flange. Simple plastic analysis of Frame A shows a 17.5 percent increase in shear carrying capacity when the location of the first hinge is shifted as described above. (A preliminary estimate of the increase for a second-order analysis is 13 to 14 percent.)

The load-deflection behavior under reversed loading shows a higher maximum load than given by the monotonic analysis. However, this monotonic analysis agrees with the experimental results of the previous frame tests when the actual locations of the plastic hinges are considered. Therefore, this significant increase in maximum load is mainly due to the residual P-Δ moments existing in the frame when the reversed loading begins.

In addition, on each of the large cycles once the deflection at the maximum load had been exceeded the load carrying capacity dropped off much slower than the monotonic analysis indicated. For the monotonic analysis this downward slope is about 3 kips/inch, whereas the experimental curve showed a slope of about 1 kip/inch. This latter effect is mainly due to strain-hardening of the steel in the plastic hinge locations.

6) Conclusions

The following tentative conclusions may be drawn from the preliminary results presented in this paper:

1. The hysteresis loops are very stable even for deformations greater than those corresponding to the maximum lateral load.
2. A considerable increase in lateral load carrying capacity over that expected from a monotonic analysis is possible.
3. Strain-hardening plays an important role in the behavior of the frames for displacements greater than those at the maximum load.
4. The presence of the residual P-Δ moments has significant effects on frame behavior and must be included in developing a rational method of analysis for repeatedly loaded frames.

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SUMMARY

The experimental behavior of two steel (A36) frames, a single-story, single-bay frame and a three-story, single-bay frame, recently tested under constant gravity loads and a program of gradually increasing amplitudes of cyclic lateral displacement is summarized. The design and the second-order elastic-plastic analyses of the test frames under monotonically applied horizontal load are outlined and comparisons with experimental results are made.

RÉSUMÉ

Le comportement expérimental de deux portiques multi-étagés en acier A36 (Equivalent à Adx charpente), portique à un niveau et à une travée, et portique à trois niveaux et une travée, récemment testés pour des charges normales constantes et pour des déplacements cycliques latéraux dont les amplitudes ont été incrémentées graduellement, est resumé. Le calcul et les analyses du second ordre dans le domaine élasto-plastique des portiques sous charge horizontales unidirectionnelles, sont présentés, ainsi que le rapprochement avec les résultats expérimentaux.

ZUSAMMENFASSUNG

Das Verhalten eines einfachen Ein-Stockwerkrahmens und eines Drei-Stockwerkrahmens, beide mit der Stahlsorte A36 ausegführt, wurde kuerzlich experimentell untersucht. Die Beanspruchung des Tragwerkes setzt sich zusammen aus vertikalen Kräften von konstanter Grösse (Graditationskräfte), und Kräften welche aus den veränderlichen, horizontalen Knotenverschiebungen resultieren. Die Konzeption der Versuchsanordnung sowie die elasto-plastischen Berechnungen zweiter Ordnung sind beschrieben und Vergleiche mit den experimentellen Resultaten wurden aufgestellt.