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In-Service Lateral Stiffness of Steel-Framed Buildings

Rigidité latérale des bâtiments avec ossature métallique sous l'effet des charges de service

Seitliche Steifigkeit von Stahlskelettbauten unter Gebrauchsbelastung

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SUMMARY

This article summarises theoretical and experimental work undertaken as part of a research project entitled «Serviceability Deflections and Displacements in Steel Framed Structures». The objective of this study is to examine the static stiffness of steel-framed structures under working loads. Experimentally this is done using a pulse load induced by a hammer blow. Numerical models of the structure include structural components, exterior cladding and interior partitions.

RESUME

Cet article représente un résumé des travaux théoriques et expérimentaux dans le cadre d'un projet «Serviceability Deflections and Displacements in Steel Framed Structures». L'objectif de ces études est d'examiner la rigidité statique des bâtiments avec ossature métallique soumis à des charges de service. Expérimentalement ceci est fait in situ au moyen d'excitations dynamiques. Des modèles théoriques sont construits qui tiennent compte d'éléments structuraux classiques, de panneaux extérieurs et de murs intérieurs.

ZUSAMMENFASSUNG

In diesem Artikel werden die Ergebnisse theoretischer und experimenteller Arbeiten zusammengefasst, die im Rahmen des Programms «Serviceability Deflections and Displacements in Steel Framed Structures» durchgeführt wurden. Ziel einer Teilstudie war die Untersuchung der statischen Steifigkeit von Stahltragwerken unter Gebrauchslast. Dazu wurde die Tragwerksantwort auf die Anregung mit einem Impulshammer gemessen. Numerische Modelle der Konstruktion berücksichtigen die Stahlbauteile, Fassaden und inneren Trennwände.

1. INTRODUCTION

Increasing adoption of limit states based approaches to the design of steel structures has tended to concentrate researcher's attentions on the need to reliably predict load levels corresponding to the attainment of the structure's ultimate static strength. Thus design is based on scientific studies that ensure a suitable margin against plastic collapse, buckling, fatigue failure etc. Although codes and specifications also call for checks at serviceability, these are usually couched in rather simple terms and little real guidance on exactly how such checks be conducted or exactly what they are intended to achieve is provided. There is thus at least the suspicion of a considerable imbalance between the qualities of design for the ultimate condition and design for the serviceability condition.

It was in recognition of this that a three-part programme of research, focussing on static deflections of steel framed buildings, funded by the European Coal and Steel Community (ECSC), was stated in late 1990. It comprised:
-Investigation of the in-service performance of steel buildings (TNO-Bouw)
-Review of existing code requirements and their basis (University of Nottingham)
-Numerical studies and consideration of design models (University of Trento)

A report [1] giving the findings of each aspect of the work has been presented to the ECSC. The content of this paper is based on the in-service performance section and is complemented by three other papers at this conference which deal with the other topics.

2 DESCRIPTION OF THE MODELLED STRUCTURE

The building investigated is located at TNO-Bouw, Delft, the Netherlands (Building #11) and may be classified as an office building. This structure is representative of many low-rise steel-framed office buildings recently built in the Netherlands. An isometric view of non-structural components (exterior cladding and interior partitions) is shown in FIGURE 1. Schematic drawings of the structural frames, in the major and minor axes, are shown in FIGURE 2.

Column supports consist of base plates bolted to the caps of existing concrete pilings, which are located in holes in an existing ground slab (subsequently filled with concrete). In the major axis most beam to column connections are provided using half-height connection plates. In the minor axis, most beams are continuous and connected to the columns using cantilevered stud beams. An endoscope was used to check cross-bracing and cross-bracing to column connections.

Exterior cladding consists of minimally reinforced light-weight concrete panels, 15cm thick with openings for doors and windows. Upon close visual inspection no evidence of panel cracking, cracks between panels or cracks at the foundation level were evident.

The first floor slab is made of lightweight precast concrete units with a cast-in-place wearing surface. The roof consists of two dimensional trusses, cross-bracing and a deep-ribbed thin-walled steel decking. Both the first floor slab and roof provide a substantial degree of in-plane shear stiffness to the structure.

All interior partitions consist of 100 mm thick gypsum board. Partitions are glued to the floor slab, no attachment, however, is provided at the lateral or top edges of the partitions to the structural frame. Interior columns are covered with fire protection materials and enveloped in wooden boxes for architectural reasons. Partitions are butted against these boxes.

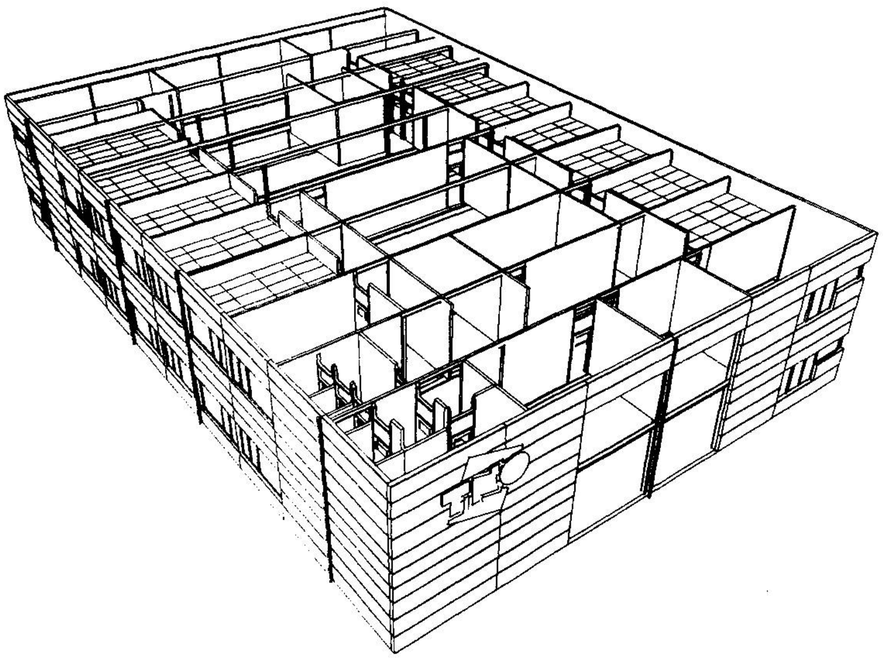

FIGURE 1: Isometrique view of non-structural components for building #11.

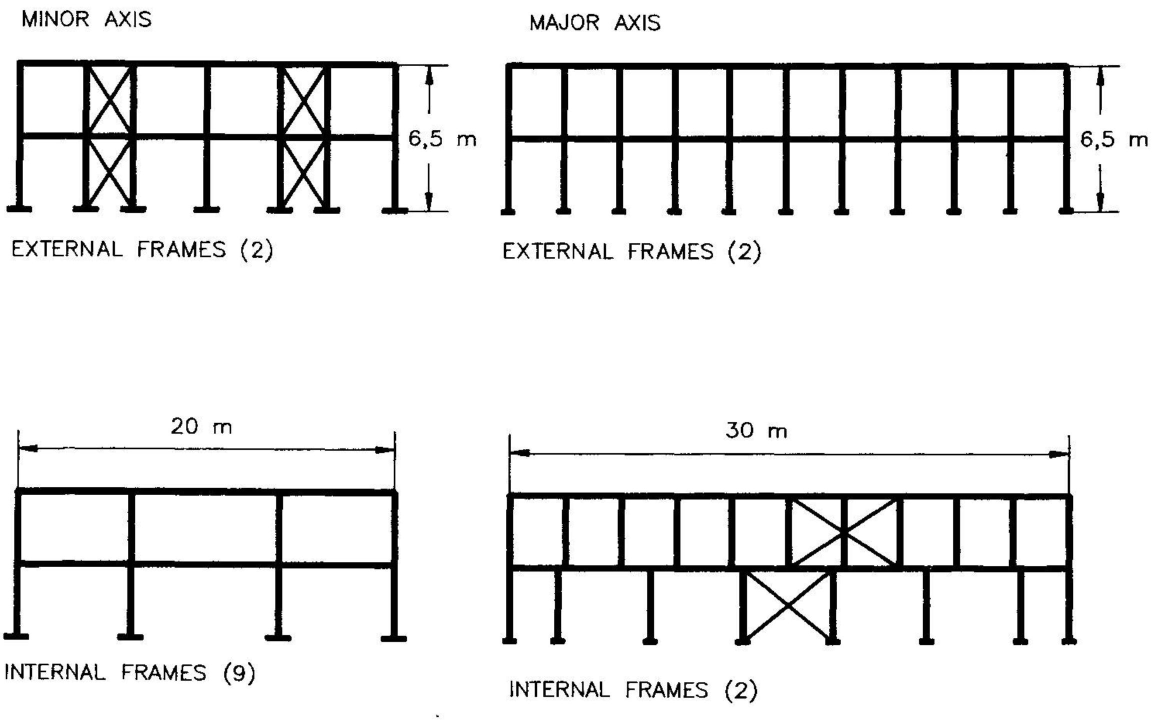

FIGURE 2: Schematic drawings of structrual frames for building # 11.

3. SERVICEABILITY MODELLING

3.1 Introduction

Significant interaction between structural and non-structural components (interior partitions and exterior cladding) may be present. The importance of this interaction depends upon the building type (office, industrial, low-rise, high-rise, etc.) the choice of non-structural components (wood, masonry, stone, glass, sandwich panels, profiled sheeting, etc.) and the type of connection between structural and non-structural components.

At service loads, some simplifications may be used to reduce model complexity. First, existing models describing the behaviour of the structural system at ultimate load levels are used. Second, non-structural components are described using their initial (un-damaged) stiffness. Past research on such components has tended to concentrated on predicting failure loads and in many cases the initial stiffness is reported. Lastly, connections between structural and non-structural components should remain un-damaged at service load levels. Connection types are categorised and their behaviours simplified (full moment connection, pin-ended, etc.). Upon close inspection no evidence of previous movements between structural frame and panel was observed. Connections have received even less attention in the literature than the in-service behaviour of non-structural components.

3.2 Modelling of building #11

Two-dimensional linear finite element structural models were constructed (major and minor axes). This is the same level of sophistication which was used for ultimate limit design calculations. Static and dynamic analyses based upon these models were performed. This provides two checks between measurements and predictions: the lateral stiffness and the lowest side-sway natural frequency.

Standard six-node beam elements and point mass elements were chosen to model structural components. Nominal section properties (cross-sectional area and moment of inertia) were used. A modulus of elasticity, E, of 210'000 N/mm2 and Poisson's ration, μ, equal to 0.3 were used to define basic steel characteristics. No plastification is expected thus plastification post yield criteria were not specified.

An upper bound estimation for cladding stiffness was assumed in the model: no movement is allowed between individual cladding panels or between panels and their foundation. Eight-node plate elements were used to model the exterior cladding. A nominal thickness of 15 cm for all elements was assumed. A modulus of elasticity, E, of 20'000 N/mm2 and Poisson's ratio, μ, equal to 0.25 were used to define basic concrete characteristics. No concrete cracking or crushing is expected. The exterior cladding is connected to the structural system (exterior columns) using structural angels (clips), which are bent to fit. Interior partitions were not included in the static model due to the lack of available connection to other components.

A nominal lateral load (equal to1N) was applied at the first floor and roof levels. The predicted values of horizontal drift were used to calculated side-sway stiffness using the following expression:

$$K = F / \delta$$

where:

K	is the side sway stiffness
F	is the applied lateral force
δ	is the predicted lateral displacement

Static and dynamic analyses are performed for the following cases:
1. The entire building, assuming full interaction between cladding and the structural frame with cross-bracing. All columns are assumed to be fixed-ended ground level. *This represents the base case from which individual parameters are studied.*
2. The base case without cross-bracing.
3. The base case with columns pin-ended at ground level.
4. The base case with all connections between beams and columns pin-ended.
5. The structural frame only (including cross-bracing).

3.3 Summary of model predictions

A detailed summary of modelling results is available in the full ECSC report [1]. Based upon the results the following observations may be made based upon the static analyses:

- *Cladding vs. the structural frame.* The model predicts that the cladding is stiffer than the structural frame.
- *In-plane vertical cross-bracing.* Cross-bracing in the structural frame has a small but detectable influence at service load levels (cross-bracing, however, often is of greatest importance during the construction phase).
- *Beam to column connections.* The influence of beam-to-column fixity (fix-ended or pin-ended) is of secondary importance.
- *Column base fixity.* The influence of column base fixity (fix-ended or pin-ended) is of no practical importance.

The results of the dynamic analyses tend to support the conclusions of the static analyses.

4. TESTING

4.1 Introduction

The building tested is the same as that described in Chapter 2 and modelled in Chapter 3. The objective of these tests is to derive in-situ values of natural frequency and static lateral stiffness. This is achieved by exciting the building using an impact hammer and measuring the response (both displacement and acceleration). This testing method has been extensively researched at TNO-Bouw. A shaker (eccentric counter-rotating masses) were used to obtain a independent check of the hammer test results.

Structural properties (static stiffness and natural frequency) can be derived by measuring the time functions of both excitation and response. Time functions are converted into frequency response functions. Comparing the frequency response functions with mathematical models, structural parameters as stiffness, mass, natural frequency and damping can be derived. The full procedure behind the derivation of structural properties is beyond the scope of this paper, but is contained in the final ECSC report [1].

4.2 Field measurements

Impact hammer field measurements were made using the field testing equipment listed in TABLE 1. Excitation was applied by means of a 10 kg instrumented hammer or a 400 kg mechanical shaker. In both cases the structure was excited on the first floor in both major and minor axis several times to obtain statistically significant values.

Theoretically when the structure is excited by a hammer impulse all frequencies have the same magnitude in the frequency domain. In practice, higher frequencies can be suppressed by mounting a rubber tip on the hammer. In this manner more energy is input at lower frequencies. The hammer was instrumented with an accelerometer, thus the impact load is determined using the hammer mass and measured accelerations.

When using a shaker, measurements were made using frequency steps (1 Hz). In this way the frequency response function could be derived and the natural frequency estimated. Applied force was not measured directly but is calculated using rotating mass eccentricity and rotation frequency.

The response of the structure was simultaneously measured on the ground floor, first floor and roof level by means of acceleration and displacement transducers in the same direction as the applied force. Typically, displacement transducers give better results in very low frequency ranges ($1 < f < 20$ Hz) and acceleration transducers give better results in the higher frequency range ($f > 20$ Hz).

Table 1: Field testing equipment.

Number	Description	Make	Type
6	Accelerometers	Sunstrand	S-700
6	Conditioner for Sunstrand	TNO-Bouw	C-S-700
6	Displacement transducers	Hottinger	B-3
6	Conditioners for Hottinger	Hottinger	KWS 3073
6	Amplifiers	Hottinger	Z 3576
1	Data acquisition system	Bakker	2570
1	Pulse hammer	TNO-Bouw	10 kg
1	Conditioner for hammer	B & K	2626
1	Mechanical exciter	TNO-Bouw	400 kg

4.3 Data reduction and test results

All signals were digitized and recorded using a data acquisition system. Afterwards the signals were converted to the frequency domain. This results in a complex frequency response function and in a coherence function between excitation and response.

For all locations and directions the natural frequency and static stiffness of the building are derived using a circle fit procedure assuming viscous damping. In the derivation of the stiffness the influence of higher order modes has been calculated. The fit procedure was carried out using the acceleration and displacement response signals.

The lateral load response, static stiffness, natural frequency and damping ratio of an entire as-build steel-framed building is thus measured using a hammer excitation. When measuring displacements response instead of acceleration response, the stiffness at ground floor level can be derived even when exciting at first floor level.

The coefficients of variation for stiffness estimations, per hammer blow, are in the order of 0.02. Average coefficients of variation (obtained by averaging all hammer blows at one specific location and direction) are approximately 0.20.

4.4 Comparison of testing methods

The determination of the natural frequency and static stiffness is potentially as accurate using a shaker as when using a hammer blow. With a hammer blow, however, measurements are both accurate and quickly obtained. Using a shaker, measurements must be made at many different speeds to obtain the buildings frequency response spectrum.

Irregardless of measurement quality, installation and measurement times for the hammer blow technique are much shorter than for the rotating mass. The rotating mass itself is heavy, requiring special equipment to move it into location. In the case of the TNO building, it was necessary to install the rotating mass using a fork lift truck through a first storey window.

As the size of the structure increases so does the necessary energy input. For very large structures it may become difficult to supply sufficient energy in the low frequency range using a hand-held hammer.

5 COMPARISON OF TEST RESULTS AND MODEL PREDICTIONS

5.1 Introduction

An existing steel-framed building was modelled and tested under service load conditions. Static lateral stiffness and natural frequency were calculated and measured. A simple finite element model was used (two-dimensional, linear and consisting of beam and plate elements). This model is similar to the model used by the designer to predict the ultimate load carrying capacity of the structural system alone. Simple assumptions are used for parameters such as column fixity, joint stiffness, cladding stiffness and the interaction between structural and non-structural components.

Test values are reported for a statistically significant population, thus the mean and standard deviation have been estimated. All comparisons are given for mean values (for one particular structure) between typical design models including non-structural components and real structural behaviour at service load levels.

5.2 Summary

A comparison of measured and predicted values are shown in TABLE 2. Finite element model predictions and test results differ by 10% to 25%. Measured values are given for a confidence level of 90%, which implies the following:
-A 10% uncertainty in the major axis.
-A 25% uncertainty in the minor axis.

Table 2: Comparison of measured and predicted values.
(Lateral load applied at the 1st floor)

Value	Test (Real structure)	F.E.M. (Real structure)	F.E.M. (Bare steel frame)
MAJOR AXIS			
Natural frequency	7.7 Hz	8.4 Hz	3.1 Hz
1st floor stiffness	640×10^6	720×10^6 N/m	99×10^6 N/m
Roof stiffness	430×10^6	560×10^6 N/m	95×10^6 N/m
MINOR AXIS			
Natural frequency	8.4 Hz	7.1 Hz	3.1 Hz
1st floor stiffness	710×10^6 N/m	650×10^6 N/m	130×10^6 N/m
Roof stiffness	420×10^6 N/m	520×10^6 N/m	110×10^6 N/m

The larger uncertainty in the minor axis is due to a rotational component of buildings response. This component was not further investigated as it could not be predicted using the two dimensional finite element model. This influence, however, is negligible when compared to the difference between the basic steel frame and models including non-structural components: between 250% and 650%. These gains can be obtained using *existing* calculation techniques and *without adding* material to the existing structure.

Experimental errors at service load levels are generally larger than those that may be expected at ultimate load levels. This is due to the participation of many building components that normally fail before ultimate loads are applied and the simplicity of the model approximating their behaviour. The implications of slightly overestimating service load level stiffness and ultimate load capacity, however, are quite different.

6. CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

An economic method of obtaining static lateral stiffness and natural frequencies at service load levels is presented: an impact hammer. Installation and measurement times of impact hammer tests are greatly reduced in comparison with shaker tests.

Measured and predicted values of lateral stiffness correspond reasonably well for the structure modelled. This is encouraging as it suggests that models used to predict the ultimate load carrying capacity of the bare structural frame may be modified to include non-structural components at service load levels. Simple assumptions can be used to define column base fixity, beam to column connection behaviour and the behaviour of connections between structural and non-structural building components. Substantial increases in lateral stiffness may be justified.

6.2 Recommendations

This work must be generalised through the testing and modelling of a range of steel-framed structures. Future testing should give preference to classes of steel-framed structures where existing codes suggest that serviceability problems may occur.

Parallel to future testing, a concentrated effort should be made to improve existing data bases containing lateral stiffness and strength limits for non-structural components commonly used in steel-framed construction. Much work remains to be done to classify and model the service behaviour of connections between structural and non-structural components.

Lastly, the results of these exercises should be used to replacing existing serviceability limits with more rational and statistically significant limits in future steel design codes.

ACKNOWLEDGEMENTS

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