DESIGN OF INDUSTRIAL STORAGE RACKS
by

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SUMMARY

The current Design Provisions for Steel Industrial Storage Racks are summarized and a progress report on a current Cornell University Research project is given.

The design of industrial steel storage racks presents several challenges to the structural engineer. Presently the design in the United States is carried out according to the 1997 edition of the Specification (1) published by the Rack Manufacturers' Institute (RMI). The RMI first published its first "Minimum Engineering Standards for Industrial Storage Racks" in 1964.

The work that resulted in the first edition of the Specification was initiated by the RMI in 1972 at Cornell University. Several editions of the Specification have been prepared based on the work by the RMI Specification Advisory Committee and the researchers at Cornell under the supervision of Professors George Winter and Teoman Peköz until 1979 and under the supervision of senior author since 1979. The RMI Specification is tied closely to the AISI Specification [2] for the provisions on Cold-Formed Steel Design. The Australian Specification for racks is based primarily on the RMI Specification with some regional enhancements. The applicable standard in Europe [3] is described in some detail in Reference [4].

The discussion in this article will be on the RMI Specification and the current research at Cornell on evolving and improving the RMI Specification. The discussion on the loads specified in the RMI Specification will be limited to Load and Resistance Factor Design. The extensive earthquake provisions of the RMI Specification will not be discussed in this article.

Most of the current research on racks in the United States is being conducted at Cornell University with the senior author as the principal Investigator for the project. The focus of the discussion in this paper is on the progress to date at Cornell University, and the conclusions may change based upon future research.

The research uses finite element solutions verified by tests on both the component and the global scale. On the component level, the topics focused upon are the behavior of joints, and the interaction of the frames with their column bases.

1. SOME FEATURES OF THE 1997 RMI SPECIFICATION (1)

1.1 Loads

Dead and live loads on racks are only a small portion of the total load on racks. Product load is a major portion of the loads applied on a rack structure. The Specification defines the product load as the products or pallet loads stored on a rack. Since the product load is usually well defined and listed on the plaques placed on the racks, lower load factors are specified for the Load and Resistance Factor Design of racks that for live loads. Below are two examples of load factors and load combinations out of eight given in the Specification:

1.4DL + LL + 1.2PL
1.2DL + 1.6LL + 0.5(SL or RL) + 1.4PL

DL and LL are dead and live loads, respectively. SL and RL snow and rain loads, respectively. PL is the product load.

Beam support connections, frame bracing, and frame bracing to column connections of racks are to be designed for horizontal loads equal to 1.5% of the factored dead load and factored product load. The horizontal forces include the effect of out-of-plumbness. The tolerance for out-of plumbness is given as 0.5 inches in 10 feet of height. The horizontal forces are to be applied separately, not simultaneously, in each of the two principal directions of the rack. The beam support connection moments are to be checked against the permissible moments (both positive and negative) determined from Cantilever Tests and/or Portal Tests (Fig. 1 and 2).

1.2 Design of Steel Elements and Members

The design of steel elements and members are carried out according to the AISI Specification [2] for cold-formed members and AISC Specification [5] for hot-rolled members. Some exceptions and modifications to these Specifications are noted in the Specification. Some of these exceptions and modifications will be discussed briefly.

The load carrying capacity beyond local buckling (post-buckling strength) is quite significant for cold-formed steel racks. For this reason some extensions to the AISI Specification had to be made.

Nominal bending moment capacity M_n is obtained by multiplying the effective section modulus S_e by the yield stress F_y . The effective section modulus S_e for perforated members is to be determined by multiplying the elastic section modulus of the net section by

$\left(0.5+\frac{Q}{2}\right)$.

This expression was obtained through reasoning that the effect local buckling would be less significant for flexural members than that for axially loaded members. The factor Q is determined by tests on axially loaded stub columns. Using this value of Q for flexure would be too conservative. The above modification factor accepts one half of Q for the compression part of the section but takes the full section for the tension part.

The interaction of lateral buckling with local buckling needs to be accounted for; thus, the expression for calculating member strength as for lateral buckling involves the effective section modulus at lateral buckling stress S_c. Modulus S_c is calculated by multiplying the net section modulus by the factor

$$1-\frac{1-Q}{2}\left(\frac{M_c/S_f}{F_y}\right).$$

In this expression M_c is the lateral buckling moment, S_f is the section modulus for the full section and Q is determined by stub column test. Though the specification does not point it out, Q could be determined by the expression

$$Q=\left(0.5+\frac{Q_{stub\,column\,test}}{2}\right)$$

as reasoned out above. The above factor is derived in accordance with the approach described in Reference [6].

As required by the AISI Specification, nominal strength of a column is determined by multiplying the column limit-state stress F_n by the effective area A_e at stress F_n. This accounts for the interaction of local and overall column buckling. Studies on the interaction of the distortional buckling and overall buckling require special consideration. A research project on this subject has just been completed. The results have not yet been published. However, it is expected that the future editions of the RMI Specification will have provisions on theis subject. The effective area is determined by the expression

$$A_e=\left[1-(1-Q)\left(\frac{F_n}{F_y}\right)^Q\right]A_{net\,\min}$$

where $A_{net\,\min}$ is the minimum cross-sectional area obtained by passing a plane through the column normal to the axis of the column. This expression is derived in Reference [6].

1.3 Frame Design

Frame design involves the use of effective length factors which are specified for various situations and the interaction equations given in the AISI Specification.

Racks in general consist of upright frames and beams connecting the upright frames. Upright frames consist of two columns braced together. There may or may not be bracing in the down-aisle direction, namely in the direction perpendicular to the upright frames.

The current practice is to do a linear analysis and account for the second order effects by a magnification factor α . For cold-formed steel racks, an interaction equation is used in the AISI Specification. The following is a simplified explanation of the AISI interaction equation. This interaction equation can be expressed as follows:

$$\frac{P}{P_n}+\frac{M}{M_n}=1$$

where P is the axial load in the column being checked determined by linear analysis. The moment determined by linear analysis M_{lin} is multiplied by the magnification factor α to obtain the second order moment M to be used in the interaction equation.

$$M=\alpha M_{lin}$$

The moment M may further be modified to account for moment gradients. The magnification factor α is defined as

$$\alpha=1-\frac{P}{P_e}$$

where P_e is the elastic buckling load of the column about the bending axis. Thus the effective length is determined for buckling about the bending axis.

The terms P_n and M_n are the limit-state axial load and bending moment, respectively, when each acts separately. For bending about the symmetry axis, as is the case for buckling in the down aisle direction, the terms P_n and M_n are determined using effective length factors K_ϕ and K_x with the nominal strength equations given in the AISI Specification.

Effective length factor for torsional buckling K_ϕ is to be taken as 0.8. The Specification gives some conditions for exceptions to this value. The effective length factor for buckling about the centroidal axis of the column section perpendicular to the aisle K_x (in general this is the axis of symmetry) can be determined either by rational analysis or taken as 1.7. The Specification gives some guidelines for determining K_x by rational analysis that it should “account for the member stiffnesses, the semi-rigid nature of the beam to column connections and the partial fixity of the base, allowing for average load reduction, as applicable.” It is noted that if K_x of 1.7 is used without analysis, then no reduction of this value shall be made

A common approach for determining effective length factors is the use of alignment charts. Some guidance for the use of the alignment charts is given in the Commentary to the Specification [5]. For racks not braced against side sway in the application of alignment charts, the beam slenderness I_b / L_b is to be replaced by $\left(I_b / L_b\right)_{red}$

$$\left(\frac{I_b}{L_b}\right)_{red} = \frac{I_b / L_b}{1 + 6\left[\frac{EI_b}{L_b F}\right]}$$

where

I_b = the actual moment of inertia of the pallet beam

L_b = the span of the pallet beam measured between the centroids of the columns supporting the beam

F = the joint rigidity determined by the Portal Test

E = modulus of elasticity

The analysis for the effective length factor for the portion of the column from the floor to the first beam level would involve the following G values as defined in the commentary of Ref. [5].

$$G_a = \frac{I_e\left(\frac{1}{L_{c1}} + \frac{1}{L_{c2}}\right)}{2\left(\frac{I_b}{L_b}\right)_{red}} \; and \, G_b = \frac{\frac{I_c}{L_{c1}}}{\frac{I_f}{L_f}}$$

where

I_c = the column moment of inertia

L_{c1} = the distance from the floor to the first beam level

L_{c2} = the distance from the first beam level to the second beam level

The fixity implied at the column base by the equation above can be calculated as

$$\frac{I_f}{L_f} = \frac{bd^2}{1440} \text{ (all dimensions in inches)}$$

$$\frac{I_f}{L_f} = \frac{bd^2}{1440(25.4)^3} \text{(all dimensions in mm)}$$

where

b = the width of the column (parallel to the flexure axis)

d = the depth of the column (perpendicular to the flexure axis)

For the above equation, the floor is assumed to be concrete, and the column connection to the floor must be adequate to develop base moments consistent with this stiffness. For other floor material the equation should be modified.

Similar rules are given for design of frames perpendicular to the aisle and for braced frames.

1.4 Tests

Several tests are prescribed in the RMI Specification for the determination of parameters that are difficult to determine computationally. Here a few of these tests will be mentioned briefly.

1.4.1 Stub Column Tests

Because of the interaction of local buckling, perforations and cold-forming effects it is necessary to carry out stub column tests on a short segment of a column to determine the behavior. These tests are carried out in accordance with the rules given in the AISI Specification. The rules are modified as applicable to the rack columns. The RMI Specification also gives rules for evaluating the test results.

1.4.2 Cantilever Tests

The test setup shown in Fig. 1 is used to determine the moment-rotation behavior of the mechanical joints of racks.

1.4.3 Portal Tests

The test setup shown in Fig. 2 is used to determine the moment rotation behavior of mechanical rack joints as well. The portal test reflects the effect of the vertical forces on the connection more accurately. However, this test is more difficult to conduct. The hinges at the column bases are prevented from moving in the plane of loading.

2. CURRENT CORNELL UNIVERSITY RESEARCH ON COLD-FORMED STEEL FRAMES

As seen in the discussion of the current RMI Specification above, the behavior and ultimate strength of a typical rack frame is characterized by many parameters. These parameters include: flexibility of beam-to-column joints, column base fixity, perforations in the columns, local buckling of member components, geometric and material imperfections and complex bucking behavior of column members. Important features and the tentative results of ongoing research are discussed below. The discussion below is a progress report on the findings and may change on the basis of the findings during the rest of the project.

The studies make extensive use of ABAQUS – a commercial finite element method (FEM) software to validate design approaches. Whenever possible the FEM solutions are compared with physical test results to gain confidence in the modeling technique.

Various types of shell elements and beam elements are used. The shell-contact element model and beam-spring element model are used to model the various features of the components and entire frame behavior.

While the shell and contact element model is accurate and reliable, it has been found that modeling of full frames by shell elements is tedious, computationally expensive and requires experienced analysts. This model best serves in validation studies (as an alternative to experiments) to evaluate the performance of other simpler numerical and analytical models.

2.1 ANALYSIS

2.1.1 Behavior of Frame Components

2.1.1.1 Beam-to-column joint flexibility:

The connection between the shelf beam and column members of pallet rack frames is generally flexible and influences the frame behavior significantly.

The current RMI specification accounts for the effect of joint flexibility on column strength by modifying the pallet beam stiffness and in turn modifying the column end restraint offered by the beam member as discussed in Section 1.3 above.

The joint stiffness is to be determined experimentally by individual manufacturer using the test setup that is mentioned in Sections 1.4 and 1.5. The specification suggests using the secant stiffness corresponding to 0.85 times the ultimate moment capacity of the joint as determined from physical tests. While the specification procedure is simple to use, there may be cases where the above assumption/simplification does not hold true. Hence, it is always rational and safe to use the correct joint stiffness value in the frame analysis by adopting joint $M-\theta$ relationship valid through the entire load history.

In the present study such expressions are developed by making use of the experimental data available in the form of moment-rotation history of a variety of joints as provided by different manufacturers. A total of 6 joint types typical to the ones used in the United States are considered in the present study. The FEM idealization used and its accuracy is illustrated in Fig. 3.

2.1.1.2 Column Base Fixity:

The column base stiffness of pallet rack frames is characterized by the base plate dimensions, number, dimensions and layout of bolts, ratio of moment to axial load at the column base and foundation characteristics. The degree to which each parameter effects the base stiffness depends on the way the column is connected to the foundation.

In the present study, the base stiffness characteristics due to bending of the base plate are studied by means of Finite Element Method. FEM idealization of the column base is illustrated in Fig. 4. Contact elements, shell elements and spring elements are used in the finite element modeling of the problem. The following observations have been made based on a large parametric study on an isolated lipped channel column member.

- The moment-rotation relationship of a typical pallet rack frame base is generally nonlinear
- The higher the axial load on the column, the stiffer is the column base.
- While the axial load increased the base stiffness by about 20% when the base plate is thin (0.25``, 6.25 mm, thinnest of plates studied), its effect is found to be insignificant in the case of thicker plates
- When the axial load on the column is accompanied by only a small amount of lateral load (1.6% of axial load), the column base stiffness is found to be very close to that of RMI specification value
- For lateral load to axial load ratios other than 0.016, the initial stiffness of column base is found to vary from 0.3-0.7 times the RMI specification value, depending on the base plate configuration and amount of axial load on the column.
- The effects of base plate configuration (plan dimensions) and number of bolts on the base stiffness are found to be negligible. However in the case of thinner plates, smaller plate configurations seem to help slightly increase the base stiffness.

2.1.2 Frame Behavior:

First, a parametric study has been carried out on four types of commonly used pallet rack frames tested at Cornell University in the 1970s. This was done to establish guidelines to prepare FEM models to study the behavior accounting for the influence of various parameters discussed above. A FEM idealization and a view of a physical test is illustrated in Fig. 5.

First the effect of warping constraint at column bases was studied. It was found that the influence of the warping constraint on the frame strength depends on the type of column members. The difference between the strengths of warping free and warping fixed cases may vary from 4% to 25%. A limited parametric study showed that the warping fixed case simulates the actual condition better.

Frames were modeled using shell and beam elements. It was found that beam elements with proper care are capable of estimating the frame strengths accurately when compared to experimental and shell element based FEM results.
Based on physical test results and FEM studies it was found that the current RMI Specification may under estimate the strengths of pallet rack frames up to about 50%. This establishes the need to review and improve the current design procedure.

While the Specification procedure has been made conservative in the absence of knowledge of the effect of various parameters on the system behavior, it is not known to what magnitude each of these parameters affect the accuracy of the design procedure.

A large parametric study involving nonlinear finite element analysis of 5 types of pallet rack frames has been carried out to quantify the conservatism of the current procedure. The factors focused on are the use of approximate effective length factors, linear beam-column interaction equation and linear amplification factor. When the column base stiffness is taken as specified in the RMI as discussed in Section 1.3 above, it was observed that:

- The strength estimates by taking $K_{\phi} = 0.8$ and $K_x = 1.7$ are satisfactory for the gravity together with lateral load cases. The calculated capacities in these cases are conservative by 10%-15%. However, for the Gravity load only case, the specification is found to be conservative by as much as 40%.
- The strength estimates by taking $K_{\phi} = 0.8$ and determining K_x from alignment charts are found to be 20-50% conservative when load on the frame is checked for either gravity or gravity together with small lateral loads. The estimates become 15-25% conservative for large lateral loads.

When the column base stiffness is taken as one half of what is specified in the RMI Specification as discussed in Section 1.3 above, it was observed that:

- Taking $K_{\phi} = 0.8$ and $K_x = 1.7$ is conservative by 10-35%.
- Taking $K_{\phi} = 0.8$ and determining K_x from alignment charts P is conservative by 20-40% when the load on the frame is either gravity or gravity together with small lateral loads and about 15-25% for large lateral loads.

In order to improve the current design procedure, several design methods using the present AISI and AISC procedures for frames and beam columns are being tried. It was found that the frame design procedure given in the AISC-LRFD Specification [5] with $K_{\phi} = 1.0$ and $K_x = 1.0$ in conjunction with either initial beam-to-column joint stiffness or secant stiffness might predict the frame strengths with sufficient accuracy. The AISC-LRFD equations can be summarized as follows:

$$\frac{P}{2P_n} + \frac{M}{M_n} = 1 \; for \; \frac{P}{P_n} \leq 0.2$$

$$\frac{P}{P_n} + \frac{8}{9}\frac{M}{M_n} = 1 \; for \; \frac{P}{P_n} > 0.2$$

$$M = \alpha M_{lin}$$

$$\alpha = \frac{1}{1 - \frac{\sum P}{\sum H}\frac{\Delta}{L}}$$

The terms used in these equations are the same as the ones defined in Section 1.3 above. Furthermore Δ is the inter-story deflection for horizontal forces determined by linear analysis, $\sum H$ is sum of all story horizontal forces producing Δ and L is the story height.

The above conclusions are based on two dimensional frame analyses. Three dimensional frame analyses are being planned for the near future. The conclusions may change on the basis of these planned studies.

2.2 Summary

The behavior of pallet rack frames with semi-rigid beam-column joints and flexible column bases is studied by experimental and numerical (FEM) investigations. A general $M-\theta$ relationship was established to model the beam-to-column joint stiffness of some pallet rack frames. The column base flexibility of rack frames caused by base plate bending is studied in detail and quantified in terms of the current specification value. Guidelines for carrying out nonlinear FEM analysis of rack frames accounting for various influencing parameters are also being studied.

A critical review of the current RMI Specification [1] was carried out. The RMI Specification was found to be conservative with regard to strength estimates. The sources of conservatism in the specification were identified. Improvements to the RMI Specification are being sought. The improvements are being sought in effective length factors, beam-column interaction equations and moment amplification factors.

3. ACKNOWLEDGEMENTS

The Cornell University research project discussed in this article is co-sponsored by the Rack Manufacturers' Institute and the American Iron and Steel Institute. This four-year project is in its third year. Dr. Kotha Rao worked on this project during the first two years as a post-doctoral researcher. Presently, Andrew Sarawit, a graduate research assistant is assigned to the project.

The support of the Rack Manufacturers' Institute and its Specification Advisory Committee chaired by Mr. Daniel Clapp, and the support of the American Iron and Steel Institute and its Subcommittee on the research chaired by Mr. James Crews are gratefully acknowledged.

The authors also wish to thank Mr. John Nofsinger of the RMI, Dr. Helen Chen of the AISI, Prof. R. Zandonini and Ms. Nadia Baldassino for their help.

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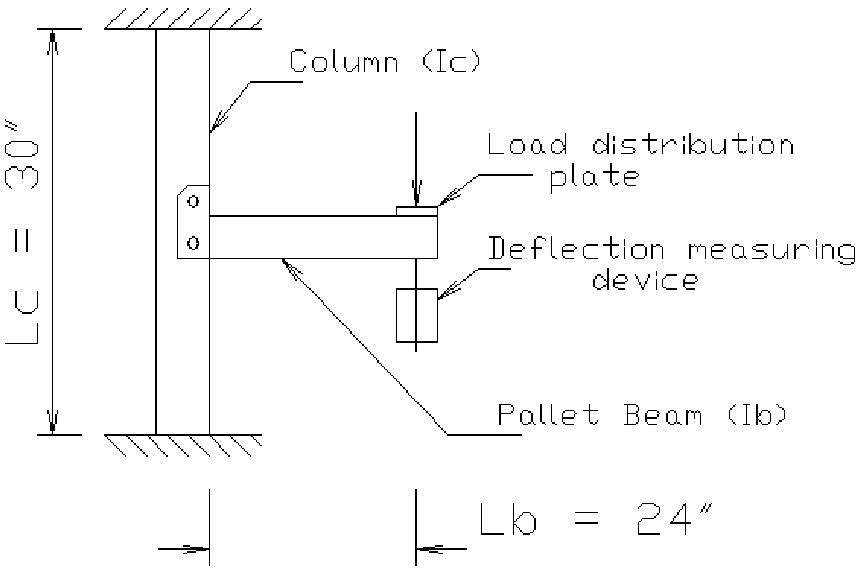

Fig. 1 Cantilever Test Setup

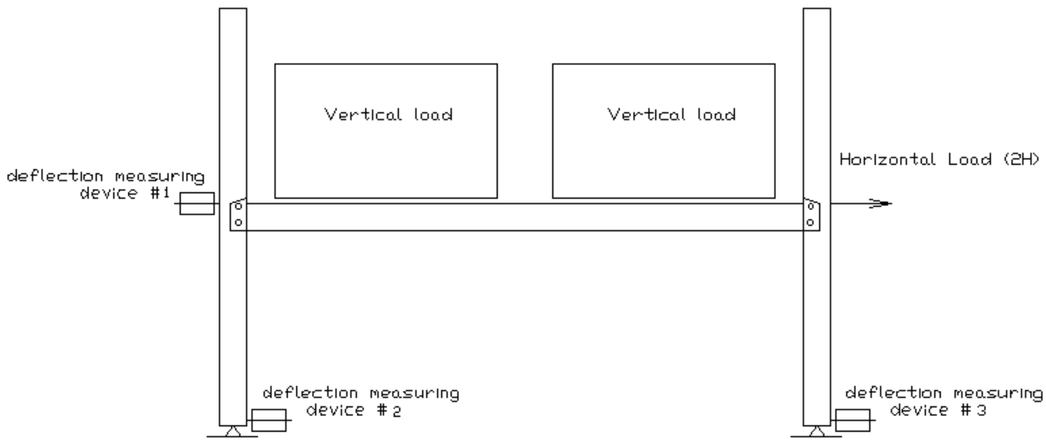

Fig. 2 Portal Test Setup

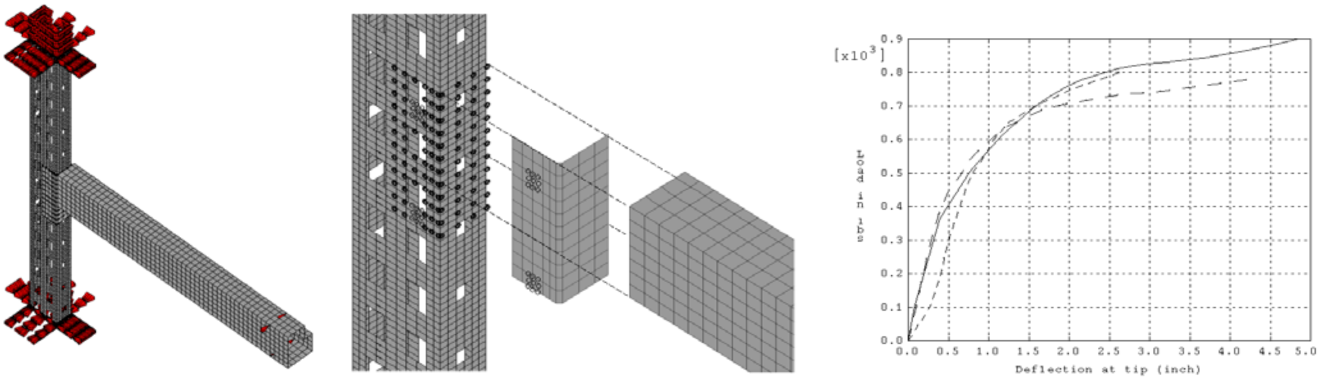

Fig. 3 Simulation of Cantilever tests to get Moment-Rotation Curves. Dashed lines are for two different ways of simulating the joints. One simulation assumes full continuity whereas the other uses contact elements to model the joint more realistically. Solid lines are for physical test results

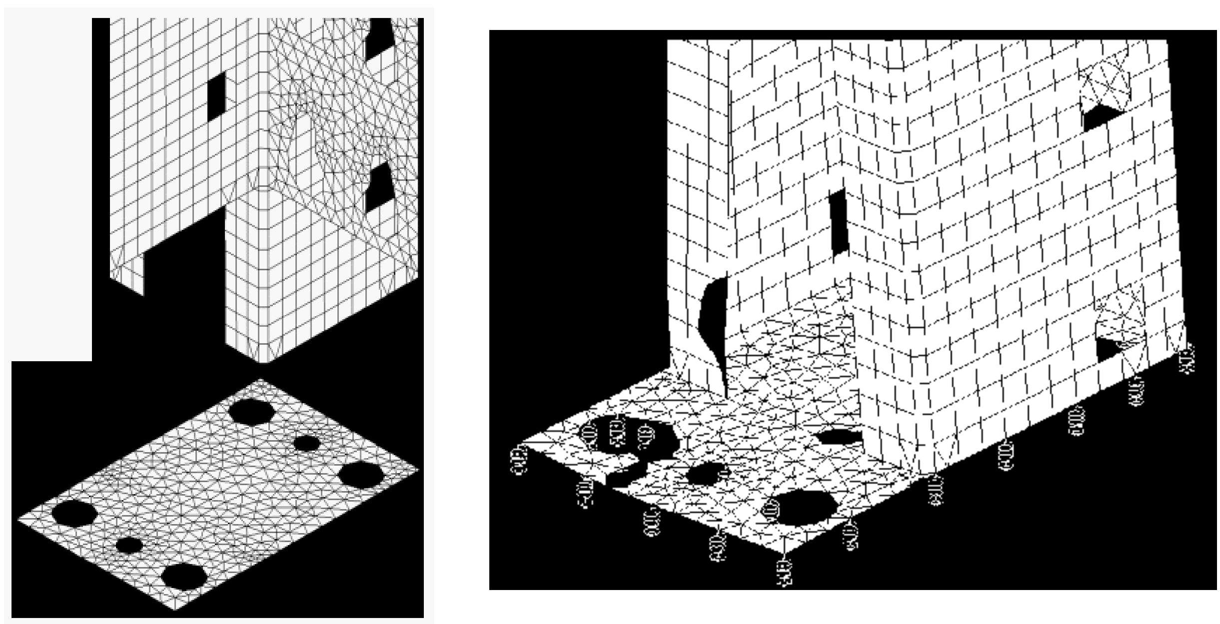

Fig. 4 FEM Simulation of Column Base

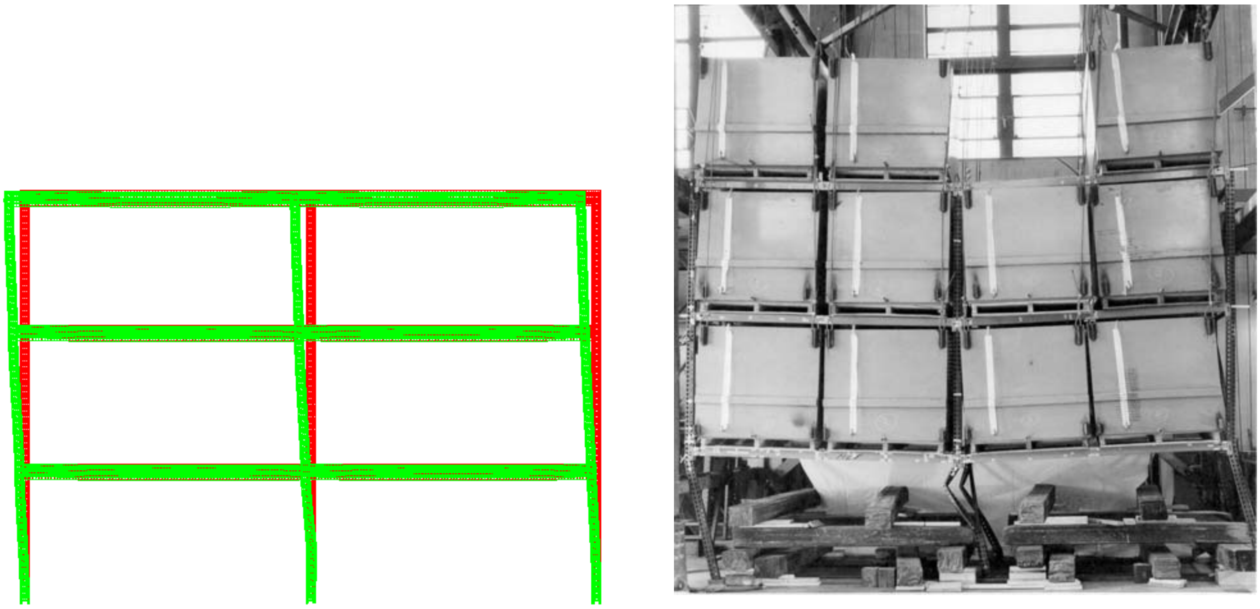

Fig. 4 Finite element simulation of a physical test on system level using shell elements