High-rise rack-supported building a unique application of cold-formed steel members

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HIGH-RISE RACK-SUPPORTED BUILDING
A UNIQUE APPLICATION OF
COLD-FORMED STEEL MEMBERS

by
C Fung *

ABSTRACT

This paper deals with the design of a high-rise automated warehouse for Simpsons-Sears Limited, constructed at Regina, Saskatchewan, Canada. The economy of a rack-supported building as well as the use of cold-formed steel members are illustrated. A method to analyze an open-section column through testing is also presented.

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INTRODUCTION

With the development of computer controlled automated storage and retrieval system, a new generation of warehouses is now emerging. Housing in these new modern warehouses is large storage facility working in conjunction with a complex storage/retrieval system. The latter consists of a number of robot-crane operating simultaneously and controlled by a single analog computer to facilitate a steady, efficient flow of goods in and out of the storage facility. The memory of the computer keeps track of each item entering or leaving the storage system. This allows goods to be stored or retrieved at random, an essential feature in warehousing a large variety of goods.

The development of high-rise construction in warehouses finds a parallel in the evolution of skyscrapers. High cost combined with shortage of serviced land have forced the construction to grow vertically. However, it was the advent of efficient stacker cranes that brought high-rise construction to warehouses, much in the same way as high-speed elevators contributed to the unprecedented height of today's skyscrapers.

The economic justification of the high-rise automated warehousing depends very much on the volume and complexity of the goods stored. In catalogue sales, the need for quick access to a large inventory of thousands of varieties of goods meets this economic justification. When Simpsons-Sears, the largest catalogue sales department store in Canada carried out its expansion in the Mid-West, a high-rise fully automated warehousing system was incorporated in its distribution centre at Regina, Saskatchewan, Canada. The construction of this complex was completed in the summer of 1977.
THE AUTOMATED WAREHOUSING SYSTEM

This multi-million dollar distribution center consists of two distinct warehousing complexes generally referred to as the low bay and the high bay. The low bay houses the conventional forklift-truck-serviced area. It handles the fast-moving items and to serve as a receiving area. Items such as appliances and carpets which have a slower cycle time, are transferred through a conveyor belt system to the high bay which houses a sophisticated automated warehousing system supplied by Clark Equipment Ltd. Cubic Storage Systems, a Division of Westeel-Rosco Limited, was subcontracted by Clark Equipment to design, fabricate and build the storage facility and the construction of the high bay.

The high bay measures 426 feet (130 m) long by 212 feet (65 m) wide by 81 feet (25 m) high. It has a total storage capacity of 26,208 pallet units and 4,104 drive-in units. Figure 1 shows the typical pallet and drive-in unit. The pallet units are supported by beams spanning between two adjacent columns. Each unit is capable of carrying a maximum load of 1000 lbs (454 kg), and is intended for the storage of appliances. The drive-in units are supported by arms cantilevered 6" (152 mm) from the face of the columns and are intended for the storage of carpet rolls weighing 1500 lbs (680 kg) maximum.

The optimum arrangement of the rack structure in relationship to the efficient use of the stacker cranes is shown in Figure 2. It calls for thirteen rows of pallet racks, accessible from twelve aisles and two rows of drive-in racks serviced by a single aisle. Based on the turnover cycle, it has been determined that a total of five stacker cranes can adequately service the entire storage facility. A transfer bay is thus required at the west end of the building to allow the cranes to switch aisles.

In order to ensure the smooth operation of the cranes, the rack structure must be erected to very close tolerances. These tolerances include not only the deviations that result from manufacturing, fabrication, shipping and erection, but also the maximum deformation of the structure under the combined loading of wind, snow and the pallet loads. Figure 3 shows the maximum deviations from the true dimensions as required by Clark Equipment (1).
THE RACK SUPPORTED BUILDING CONCEPT

The conventional concept of having a building to house the storage rack requires the design of a roof structure capable of spanning the entire 212 ft (65 m) width. It also requires 81 ft high columns to support the roof structure. The alternative is to introduce interior columns to reduce the span of the roof structure. While the first alternative is neither practical nor economical because of the unusually large span and height, the second alternative is not functional. The presence of interior columns does not only waste a certain percentage of the valuable storage volume, but also interferes with the operation of the stacker crane.

The concept of a rack-supported building is then apparent. By placing the cladding of the building directly on the rack structure, the need of building columns is eliminated. The saving in material and labour achieved by integrating the two structures is further evident by comparing the building loads (5) to the pallet loads. Table 1 illustrates this comparison. Since the sway force is not assumed to act simultaneously with the wind force (6), the rack structure can be said to have already been designed for the wind. As for the gravity load, the roof load is only a small percentage of the pallet load. The rack columns being braced at frequent intervals can support this additional load without a significant increase in material. The biggest saving comes from the drastic reduction in the size of the roof structure. With the columns closely spaced in both directions, a light weight cold-formed steel joist is what is needed to carry the load. Considerable saving was confirmed by the preliminary design carried out by Carruthers & Wallace, Consultants Ltd of Toronto.
WHY COLD-FORMED STEEL MEMBERS.

Unlike hot-rolled sections, cold-formed steel members offer the designer the freedom of designing a section tailored to his specific need. In addition he has the choice of the strength of the steel, the exact thickness as required, the geometry of the section to give the highest desired property and the method of manufacturing to the desired tolerances. These choices are particularly important in a situation where large quantities of members are identical or similar in cross-section. By manipulating the four choices, the section can be optimized to achieve maximum saving in material and/or labour.

In order to obtain a better appreciation of this design freedom, some statistics of the member quantities are in order. Table 2 shows the quantity for the major members in the structure. In spite of the varieties only two lipped channel sections are required to cover all the needs adequately. The sections are shown in Figure 4. Channel sections are favoured because of two features unique to the section. First, the channel shape being an open section is easy to roll-form. Second, the web of the channel facilitates a simple bolted connection. Holes can be pre-punched in the shop with high accuracy. Punching operation is simple and fast and can be incorporated into the roll-forming process resulting in a saving in shop labour.

For convenience of discussion, the members shown in Table 2 are re-grouped into two categories, namely the compression members and the flexural members.

Compression Members

The primary member in this category is the column of the rack structure. There are three criteria the column section is designed to meet:

1. The dimension of the column parallel to the aisle must be kept to a minimum,
2. Support roof, pallet and wind or sway load using minimum material,
3. Facilitates a simple self-gauging bolted connection for connecting the beams in the field.
The final geometry is shown in Figure 4 (a). The physical meaning of above criteria is illustrated in Figure 5. Since there are 109 columns in one row of racks, an inch increase in each column would result in an increase of 9 feet in the overall length of the building. The need to keep the dimension to a minimum is self-apparent. The section is roll-formed from 50 ksi (345 MPa) yield steel with thicknesses ranging from 0.105 in (2.7 mm) to 0.168 in (4.3 mm). The outside dimensions are kept constant. This allows the amount of material be proportional to the loading at various heights of the column. The allowance for beam connection has a profound influence on the selection of the channel shape. There are 45,760 beam-to-column connections to be made in the field under various inclement weather of the winter and at a considerable height from the ground. In addition, the beams must be placed within 0.060 in (1.52 mm) of the true elevation. The open shape provides the simplest solution - a bolted connection with holes pre-punched in the shop at predetermined locations to an accuracy of 0.030 in (0.76 mm). As mentioned earlier, punching operation can be automated resulting in a considerable saving in labour.

**Flexural Members**

Flexural members include beams, purlins and girts. The criteria governing the design of the section is less complex. The prime consideration is to produce a maximum moment of inertia to weight ratio. Since moment of inertia changes with the fourth power of the depth of the section, it is more beneficial to vary the depth than thickness. All sections used in this project were rolled from 0.074 in (2.00 mm) thick, 50 ksi (345 MPa) yield steel. One special feature (see Figure 4(b)) of the section is the ability to snap together to form a box section to give higher load carrying capacity and better appearance.
TESTING OF THE OPEN-SECTION COLUMN

The exact analysis of the torsional stability of an open-section column is often too complex for routine design purposes. The latest edition of Canadian design standard for cold-formed steel members CSA S-136 (2) provides the following formulae for torsional-flexural buckling design:

\[
F_{st} = \frac{1}{2B} \left[ F_s + F_t - \sqrt{(F_s + F_t)^2 - 4BF_sF_t} \right] \tag{1}
\]

when \( F_s = \frac{151000}{KL_x} \) \tag{2}

\[
F_t = \frac{0.52}{Ar_o^2} \left[ 11300J + \frac{291000}{(KL_g)^2} C_w \right] \tag{3}
\]

However, these formulae are derived for a member with simple end conditions and the unsupported length \( L_x, L_y \) and \( L_g \) are equal. The boundary conditions of the rack column are not compatible with conditions of the formulae listed above. The rack column is braced at frequent intervals in the plane of symmetry and at larger intervals in the plane perpendicular to it, i.e. \( L_x \neq L_y \). Furthermore, the exact warping constant \( C_w \) for the unusually shaped column section is too complicated to be calculated. There is also the question of the effect of the moment present in the plane of symmetry. For these reasons, the load carrying capacity of the rack column was determined by full-scale tests described below.

1. Single Column Tests (3)

The purpose of the test series is twofold: first, to compare the capacities of the unreinforced and the reinforced sections, and, second, to provide a base for the evaluation of the frame assembly test results. The reinforced section is an attempt to convert the open-section into a close-section, thus eliminating the torsion effects. The reinforcement consists of a 0.074" formed channel fitted over the open side of the section and welded to it. The reinforcement does not cover the full length of
the column. It terminates 10" from each end.

Various lengths of column were loaded axially to failure in a test set-up shown in Figure 6. The load was applied concentrically through a ball-joint end seat. The ball joint seat was designed to allow end rotation as well as twisting thus defining accurately the unsupported lengths for x, y and θ axis.

Test results are shown in Table 3 and graphically presented in Figure 7. A safety factor of 1.9 is applied to the results which are compared with the allowable stresses as defined by Clause 5.6.2 for flexural buckling and 5.6.3 for torsional-flexural buckling in the CSA S-136. The discrepancy between the test results and the theoretical values for the unreinforced section is attributed to the inaccuracy in the calculation of the warping constant. It is most interesting to note that the reinforced section behaves practically like a close-section reflecting the effectiveness of the reinforcement.

2. The Frame Assembly Tests (4)

The purpose of this test series was to establish the capacity of the unreinforced column section in the frame assembly. Two typical frame configurations were tested. They are shown in Figure 8.

Frame A represented a typical section in the pallet rack. The unsupported length in the plane of the frame (Lx) was about the same as that in the plane perpendicular to the frame, i.e. Ly. Moments were induced in the column by loading the beams to the design capacity with concrete blocks. The beams were loaded in a checkerboard pattern to produce a single curvature in the columns under test. Loads were applied concentrically to the columns through the ball-joint seats as in the single column tests.
Sidesway was prevented by a set of diagonal bracings. Two different panel heights, 64.5 in (1638 mm) and 88 in (2235 mm) were tested. Frame B represented a typical section in the drive-in rack. The unsupported lengths in the plane of the frame ($L_y$) and in plane perpendicular to the frame ($L_x$) were 99 in (2515 mm) and 148 in (3760 mm) respectively. Two tests were conducted. In the first test 1500 lbs (6.67 kN) on a pallet were placed on each pair of arms. During the test it was observed that pallet was acting effectively as a tie thus reducing the effective length. The ultimate load on the frame was 58800 lbs (262 kN). In the second test, it was decided to remove the pallet loads. A lower ultimate load of 43400 lbs (193 kN) was obtained. This confirmed the effect of the pallet.

The results of the frame tests are listed in Table 3. After divided by the total sectional area and a safety factor of 1.9, the stresses thus obtained are plotted on the curve shown in Figure 7. In comparing with the single column test results, the following observations can be made:

(a) The difference between the results are small and can be attributed to the effect of the moments present in the frame tests.

(b) The bracings in the plane of symmetry of the section provide sufficient restraint to prevent the twist of the section at the panel points. Therefore, $L_y$ is equal to $L_y$ which should be used in equation (3).

(c) In lieu of calculation, single column tests can be used to establish the allowable axial stresses. The stresses thus obtained can be applied to the interaction formula for combined bending and axial stresses.
THE STRUCTURE

The entire structure consists of 1020 vertical trusses field-bolted together with beams at 14 levels. Each vertical truss is a shop-welded assembly composed of two columns separated by a series of web members. The assembly is welded together in a jig to achieve the desired tolerances.

The purpose of this welded construction is twofold. Because of the size, the members are too flexible to be handled individually. By welding the members into a structural assembly, it forms a rigid, stable unit which can then be handled, shipped and erected with a minimum of labour. Structurally, the web members serve as bracings to prevent the columns from buckling in the weak axis. The assembly as a unit is utilized as a vertical cantilever truss to resist lateral forces.

The steel roof deck is 0.036" (0.95 mm) thick and 1.68" (43 mm) deep with ribs at 8" (203 mm) on centre. The deck is continuous over four purlins spaced at 110" (2794 mm) apart. The purlin is a 6" (152 mm) deep coldformed channel section as shown on Figure 4(b). The steel decks together with the purlins are designed to act as an horizontal diaphragm to distribute the lateral forces to the vertical bracing systems, thus achieving the highest stability as in a closed rectangular box. While the vertical cantilever trusses are sufficiently rigid to stabilize the structure in the transverse direction, the rigid frame formed by the beams and columns in the longitudinal direction (i.e. the direction parallel to the aisle) is not adequate to resist the lateral forces. As a result, two sway towers designed to resist the lateral forces transmitted through the roof diaphragm have been incorporated into the side walls of the building (Figure 9). It is interesting to note that in the longitudinal direction it is the 1.5% sway force requirement as stipulated in Reference (6), that governs the design. In the opinion of the writer, this arbitrary requirement should be re-examined in the light of other building structural requirements.
CONCLUSIONS

A high-rise rack-supported building has been successfully built entirely from cold-formed steel members. The use of two custom-rolled channel sections as an economic solution for building a large structure to close tolerances has been demonstrated. Although existing design codes do not adequately cover the design of channel sections, testing can ensure its proper use. This project also demonstrates the economy of rack-supported buildings and the use of cold formed steel members to build large structures.

ACKNOWLEDGMENT

The design of the special foundation and the preliminary study of the superstructure of the high bay were done by Carruthers and Wallace, Consultants Limited of Toronto, Canada. The testings were conducted by Dr R G Drysdale of McMaster University at Hamilton, Canada. The writer is grateful to Cubic Storage Systems for permission to release the test data and to his engineering colleagues at Westeel-Rosco Limited, Canada, for their contribution to the success of this project.
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SYMBOLS

A  Cross sectional area \((\text{in}^2)\)
B  Constant \(= 1 - \left(\frac{x_0}{r_0}\right)^2\)
C_w Warping constant of torsion of cross section
F_{st} Allowable stress for torsional-flexural buckling (ksi)
J  St Venant torsion constant for thin wall
K  Effective length factor
L_y Unsupported length in the plane of symmetry of a single
    symmetric open-section member (in.)
L_x Unsupported length of the member in the plane perpendicular
    to plane of symmetry (in.)
L_A Rotational unrestraint length of member (in.)
r_o Polar radius of gyration of cross-section about the shear
    centre (in.)
x_o Distance from shear centre to centroid along x-axis.
TABLE 1 DESIGN FORCE COMPARISON

<table>
<thead>
<tr>
<th></th>
<th>Building</th>
<th>Rack</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roof Load</td>
<td>3.2 (k)</td>
<td>-</td>
</tr>
<tr>
<td>Paint Load</td>
<td>-</td>
<td>26.0 (k)</td>
</tr>
</tbody>
</table>

\(N-S\) Direction
- (Wind Load) 624.0 \(k\) -
- (Sway Load) - 485.0 \(k\) -

\(E-W\) Direction
- (Wind Load) 343.0 \(k\) -
- (Sway Load) - 485.0 \(k\) -

Total per Frame

TABLE 2 MEMBER QUANTITIES

<table>
<thead>
<tr>
<th>Member</th>
<th>Length</th>
<th>Quantity</th>
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</thead>
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<tr>
<td>Rack Col</td>
<td>80'6&quot;</td>
<td>1990 pcs</td>
</tr>
<tr>
<td>Rack Beam</td>
<td>18'4&quot;</td>
<td>3280</td>
</tr>
<tr>
<td>Purlin</td>
<td>19'3&quot;</td>
<td>706</td>
</tr>
<tr>
<td>Girt</td>
<td>18'4&quot;</td>
<td>910</td>
</tr>
</tbody>
</table>
### TABLE 3  SUMMARY OF TEST RESULTS

<table>
<thead>
<tr>
<th>TEST NO.</th>
<th>THICK (IN.)</th>
<th>L_y (IN.)</th>
<th>L_x (IN.)</th>
<th>BEAM LOAD (k)</th>
<th>ULTIMATE LOAD * P_u (k)</th>
<th>ALLOWABLE STRESS f_a = P_u /1.9A (ksi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>C-1</td>
<td>0.168</td>
<td>40.75</td>
<td>40.75</td>
<td>-</td>
<td>76.4</td>
<td>23.9</td>
</tr>
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<td>C-2</td>
<td>0.168</td>
<td>69.25</td>
<td>69.25</td>
<td>-</td>
<td>52.9</td>
<td>16.6</td>
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<td>C-3</td>
<td>0.168</td>
<td>92.75</td>
<td>92.75</td>
<td>-</td>
<td>38.1</td>
<td>11.9</td>
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<tr>
<td>C-4</td>
<td>0.168</td>
<td>148.75</td>
<td>148.75</td>
<td>-</td>
<td>16.9</td>
<td>5.3</td>
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<tr>
<td>CR-1</td>
<td>0.168</td>
<td>69.25</td>
<td>69.25</td>
<td>-</td>
<td>68.8</td>
<td>21.6 )</td>
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<tr>
<td>CR-2</td>
<td>0.168</td>
<td>92.75</td>
<td>92.75</td>
<td>-</td>
<td>58.9</td>
<td>18.5 ) Reinforced</td>
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<tr>
<td>CR-3</td>
<td>0.168</td>
<td>148.75</td>
<td>148.75</td>
<td>-</td>
<td>30.0</td>
<td>9.4 )</td>
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<tr>
<td>FA-1</td>
<td>0.168</td>
<td>64.5</td>
<td>64.5</td>
<td>4.0</td>
<td>96.7</td>
<td>15.1</td>
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<tr>
<td>FA-2</td>
<td>0.105</td>
<td>88.0</td>
<td>88.0</td>
<td>3.6</td>
<td>46.3</td>
<td>11.3</td>
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<tr>
<td>FA-3</td>
<td>0.105</td>
<td>99.0</td>
<td>148.0</td>
<td>1.5</td>
<td>58.8</td>
<td>14.3</td>
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<tr>
<td>FA-4</td>
<td>0.105</td>
<td>99.0</td>
<td>148.0</td>
<td>0.0</td>
<td>43.4</td>
<td>10.6</td>
</tr>
</tbody>
</table>

* For Frame Tests FA-1 to FA-4, Ultimate Load Applied to Two Columns
FIG. 1
HIGH-RISE RACK-SUPPORTED BUILDING

(a)

(b)

FIG. 4

FIG. 5
FIG. 6

(a) Unreinforced

(b) Reinforced
FIG. 7
64.5" or 88.0"

FIG. 8