Analysis of steel-structure/masonry-wall interaction in historic buildings

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ABSTRACT: Conservation of historic high-rise buildings requires analysis of the peculiarities of early modern construction. The beams, columns, and facades of large steel-frame, masonry-wall buildings experience complex undesigned interactions. Masonry curtain walls of these buildings are usually 300 mm thick, capable of resisting structural loads. American investigation of interactions between masonry curtain walls and steel frames began with facade inspection laws intended to find damaged masonry on high-rise buildings. Structural effects found include cracking, out-of-plane displacement, and spalls resulting from thermal stress, sidesway, and rust-jacking. This paper compares the effect on masonry curtain walls of various structural mechanisms, specifically for high-rise buildings as constructed in the United States between 1900 and 1930. For each mechanism, the stress and strain interaction of steel and masonry elements is considered, rather than analyzing them separately as in modern design practice.

1 INTRODUCTION

As the conservation movement has expanded to include modern buildings, and as the oldest steel-frame high-rise buildings have passed one hundred years since their construction, engineers have begun to study technical issues in maintaining and repairing historic high-rises. One of the most difficult issues is the interaction under load of heavy masonry curtain walls and steel frames; since this was neither anticipated in the original designs nor is common in the construction of current buildings. Much of the damage that is seen in these buildings – including cracking, out-of-plane displacement, and masonry spalling – is similar in appearance to that seen in traditional masonry construction but results from different causes. The causes can best be explained by analyzing these buildings as examples of a unique type.

New York City has been a center of study of historic high-rises, both formally within the conservation community and empirically by engineers and architects inspecting and designing repairs for facades. The historic reason for the study is the sheer concentration of such buildings in the city: in 1929, there were 2479 buildings in New York higher than ten stories out of 4829 in the United States. Chicago had the second greatest total, 449 (Regional Survey, 1931). The empirical reason for the study is the 1980 city law that requires periodic inspection and repair of all tall-building facades. Recognition of New York's unique tall-building history has not necessarily translated into conservation and restoration techniques specific to the buildings. Simply put, architectural histories, technical descriptions of designated landmarks, and ad hoc conservation knowledge among designers and contractors focus more on the masonry envelopes of those buildings than their structural details. At one extreme, descriptions of tall buildings in architectural guides may omit critical details of a building, such as one description of the 320 m Chrysler Building that fails to mention that it has a brick-veneer exterior wall, let alone that it has a steel frame (White & Willensky, 2000). At the other extreme, technical literature specifically written for conservation typically emphasizes materials concerns over less-known structural ones. The New York Landmarks Conservancy, a non-profit advocacy and technical expertise organization, has created a useful guide to the repair of building facades that emphasizes work on high-rise buildings (Meadows, 1986). The guide correctly mentions steel deterioration and the effects of wind pressure and thermal changes as causes of damage, but is aimed at identifying and repairing materials damage. Unfortunately, there is no equivalent guide for identifying and repairing damage caused by interaction between steel and masonry elements, or, as they are often identified, structure and architecture. The lack of technical sources is exacerbated by the visual similarity between material deterioration and secondary damage from structural movement. For example, brick faces may spall because of incompatible pointing (a materials problem) or because of high compressive stress from...
lack of expansion joints (a structural problem). Structural problems such as excessive sidesway are typically not visible in themselves, but become manifest in damage to non-structural elements.

2 HISTORICAL AND ARCHITECTURAL CONTEXT

The earliest steel-framed buildings in the United States, completed before building codes recognized the reality of curtain-wall construction, had very thick exterior walls, with 600 mm of masonry common in the 1890s. A trend towards 400 mm walls shortly before 1900 was finalized by the 1901 New York City Building Code, which required minimum 300 mm walls on all steel-frame buildings. Walls continued to be built thicker than minimum in order to carry architectural ornament. Various facing materials were used, most often brick and glazed terra cotta.

Steel-frame buildings constructed before the 1901 code vary greatly in structural type, and can be considered as partly experimental. Those constructed after 1901 and before the 1929 economic crash are much more structurally uniform, as described below.

Figure 1. 953 Fifth Avenue is the narrow building with a three window-wide street facade, directly above the truck.

This latter class of building is highly concentrated in New York and Chicago, as no other city had a large number of tall steel-frame buildings constructed in that era, and construction details from the post-World War II period are significantly different.

The character of large portions of Manhattan is marked by early twentieth century steel-frame buildings: the downtown and midtown business districts contain hundreds of office buildings of this type, while the north-south avenues of the uptown residential neighborhoods are lined with miles of mid- and high-rise apartment houses. Some of these buildings, such as the 240 m Woolworth Building of 1913, were planned as architectural monuments and have remained so since, but most of these buildings were meant as ordinary “background” architecture (Fig. 1). Individual buildings have been designated as protected landmarks by the New York City Landmarks Preservation Commission. There are also designated landmark districts containing many of these buildings, but the significant buildings within the districts (such as the Upper East Side district) are often low-rise, traditional-construction buildings such as churches and private residences. Despite the fact that so many steel-frame mid- and high-rise buildings in Manhattan are not considered architecturally distinguished, this type of building is of great interest because it quantitatively dominates use and repair.

3 BUILDING DESCRIPTION

The analysis in this paper concerns the structural effect of outside forces on all buildings of a given type – steel-frame high-rises built between 1900 and 1930 – using numerical examples from one such building. Generalizing from the analysis of one building to a class depends greatly on the assumption that the class has strongly marked properties that can be defined in advance. The author's observations during restoration projects and the historical record both confirm this assumption – the existence of what architectural historian Carol Willis has called “vernaculars of capitalism,” where building codes, street layouts, and local economic conditions combined with the standard building technology of the day to produce “standardize[d] highrise design” (Willis, 1995). The description that follows is based largely on the author’s observations, but is similar to those in traditional architectural and technological histories such as Elliott (1992) and Condit and Landau (1996).

3.1 Characteristics of the general type

The defining characteristic of the type is the presence of a structural-steel skeleton frame designed to carry all gravity and lateral loads. Under the building codes in force during the period of interest (the first thirty
years of the twentieth century), the only lateral load explicitly used in the design of multi-story apartment, office, and industrial buildings was wind load; under current codes in the United States, wind load usually governs for steel-frame buildings except in the high-seismic areas (primarily the west coast) where relatively little high-rise construction took place until after 1945. Connections were typically riveted except for beam-to-girder double-angle connections, which were either riveted or bolted.

The most common lateral-load systems were moment frames with semi-rigid bracket connections, often top and bottom stiffened angles. Knee braces or more complex moment frames were used on slender or unusually tall buildings; full-bay bracing was used only in the tallest and most slender buildings. The reliance on moment frames is in part an artifact of design methods: the use of portal- and cantilever-frame analyses did not provide accurate lateral drift results that might have encouraged the use of stiffer frames. Matrix-based analysis was impractical without computers and moment-distribution was not yet available (Cross, 1930).

Several floor systems were in use simultaneously during the period of interest. In 1900, terra-cotta tile arches were the standard method of providing a floor between beams. By 1930, the most common system was the draped-mesh slab, often constructed using cinder-aggregate concrete. Ordinary bar-reinforced concrete slabs were sometimes used, although they were rare in New York during the 1920s and 30s. Patented reinforced-concrete slabs, such as the Kahn System, were most common in the 1910s, but appear throughout the period.

Building facades were solid masonry, consisting of a veneer of ashlar, terra cotta, or face brick over common-brick back-up. The 1901 New York Building Code cleared the way for walls of constant 300 mm thickness by explicitly recognizing the frame structural support of the walls, in place of the codes based on masonry structure previously in use. The detail used for supporting the back-up masonry nearly always consisted of the masonry resting at each floor on either the spandrel beams or the slab above the spandrel beams. The veneer was commonly supported through mechanical interlock with the back-up (e.g., headers). Windows were simple rectangular openings with either loose lintels or, rarely, hung lintels. Masonry piers built integrally with the walls were typically used to provide fire-protection to the spandrel columns.

The most important structural aspect of these buildings is not obvious during cursory examination. Unlike modern construction, where great efforts are made to structurally isolate facades through the use of expansion joints and flexible ties, the exterior of these buildings is a system of masonry and metal elements in continuous contact. More specifically, there are no expansion joints of any kind in the curtain walls and the fire-proofing piers tie the columns to the walls. The presence of these piers and the close contact between masonry, spandrel beams, and floor-slab edges makes independent movement of the walls and frame impossible, and therefore negates a common design assumption.

3.2 Case study: 953 Fifth Avenue

The apartment house constructed in 1924 at 953 Fifth Avenue in Manhattan is typical structurally except for its bar-reinforced concrete slabs. At fourteen stories (46 m) above grade, it was not particularly tall when built, however it was built on a single 7.6-meter-wide lot and it therefore has a fairly high slenderness ratio of 6 (Fig. 2). In reality it receives no wind load in the

Figure 2. Typical floor framing plan, 953 Fifth Avenue. 1 is a spandrel column in a masonry pier, 2 is the typical floor slab, and 3 is a typical spandrel beam embedded in the wall.
short plan direction because it is sheltered on both the north and south sides by buildings of similar height and age, but it was, of course, designed as free-standing.

953 Fifth has always been a high-end apartment house. There is little difference in structure between luxury and ordinary buildings; the use of limestone veneer on the west (street) facade being the only noticeable departure from ordinary materials.

There is a light court at the south-east corner that is roughly half the east-west length of the building and one-third its north-south width. The basic frame consists of seven single-bay, knee-braced frames spanning north-south, connected to create three multiple-bay moment frames running east-west.

Two conditions were analyzed: the north facade, which is a wide and windowless lot-line wall, and the west facade, which is a narrow and slender windowed wall. Both walls are 300 mm of solid masonry, with all-brick construction on the north and limestone veneer over brick back-up on the west. The column centerlines are 356 mm back from the wall faces, with the spandrel beam locations varying to provide 100 mm of cover between the flange tips and the wall faces (Fig. 3). Commercial finite-element software (Dr. Frame version 2.0) was used to analyze the bare steel frame and the frame with masonry shear walls, and classical analysis was used for the walls as vertically-cantilevered beams.

3.3 Summary of analysis

Table 1 lists the movements associated with various forms of motion in the building. All loads are the maximum design loads (full dead and live load on interior floors, full wind load on exposed wall area, full thermal variation) unless otherwise noted. The details of each condition are described in the following text.

Several patterns are apparent in the calculated movements and described in detail below. First, load sharing between the frame and walls will occur under initial dead load and under some wind loads. Second, the slenderness of a given facade is a determining factor in load sharing from differential stiffness. Third, significant qualitative differences in results exist depending on whether full code loads are used or reduced loads that reflect more ordinary circumstances.

4 EFFECTS OF TEMPERATURE CHANGE

New York has large temperature swings every year, with winter average lows of \(-3^\circ C\) to \(-1^\circ C\) and a record low of \(-26^\circ C\) and summer average highs of \(27^\circ C\) to \(29^\circ C\) and a record high of \(41^\circ C\). The standard temperatures for HVAC design are \(-11^\circ C\) for winter and \(32^\circ C\) for summer. Because of direct thermal gain from sunshine on masonry, it is common for masonry temperatures during summer days to exceed 38°C, therefore a maximum temperature swing of 32°C was used for examination of facade thermal effects. The masonry wall is assumed to have been built at the mean

<table>
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<th>Cause</th>
<th>Location</th>
<th>Structure</th>
<th>Movement (mm)</th>
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<td>Frame</td>
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</tr>
<tr>
<td>Sidesway</td>
<td>North wall</td>
<td>Frame &amp; masonry</td>
<td>0.05</td>
</tr>
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<td>Masonry</td>
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<td>Masonry</td>
<td>4.8*</td>
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<td>Thermal</td>
<td>North wall</td>
<td>Masonry</td>
<td>7.4**</td>
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<td>Compression</td>
<td>North wall</td>
<td>Column (live + dead)</td>
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<td>Masonry (live + dead)</td>
<td>9.7</td>
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<td>Masonry (dead)</td>
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</tr>
<tr>
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<td>Frame</td>
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</tr>
<tr>
<td>Sidesway</td>
<td>West wall</td>
<td>Frame &amp; masonry</td>
<td>6.1</td>
</tr>
<tr>
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<td>Masonry</td>
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<tr>
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<td>West wall</td>
<td>Masonry (reduced wind)</td>
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<tr>
<td>Thermal</td>
<td>West wall</td>
<td>Masonry</td>
<td>1.3*</td>
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* Horizontal movement.
** Vertical movement.
temperature of 11°C, creating equally large thermal stresses at the minimum and maximum outside temperatures; for the sake of simplicity, the effects will be discussed for expansion only although equal effects exist for contraction.

The masonry of the facade is directly exposed to both the overall ambient temperature changes and (depending on orientation) direct heat gain from the sun. Buildings of this type have no dedicated insulation, but rather rely on the insulating properties of the masonry and the interior plaster. Since the columns are encased in masonry piers of equal exterior and interior thickness and the spandrel beams are encased on the exterior with masonry and on the interior with concrete fire-protection, the steel temperature is likely to be roughly half-way between that of the interior and exterior air. This tends to reduce the effects of extreme temperatures but this ameliorative effect is not considered here.

A wall plane exposed to a large temperature change will undergo both vertical and horizontal movement. The effect of vertical thermal expansion is limited by the geometry of the connection between wall and frame: since most of the wall is back-up masonry that is vertically bounded by spandrel beams, and the veneer is regularly tied to the wall by header bricks, any vertical expansion of the wall will be resisted by the spandrel beams and the forces transmitted to the columns. Vertical motion is ultimately not damaging because, pushing upwards from the foundation, it can simply lift the unrestrained upper portions of the facade and adjacent steel upwards. At worst, this creates differential movement between spandrel and interior columns, but this is resisted by the ductile-metal frame.

Horizontal thermal expansion is potentially more serious, because it is not resisted by gravity but it is restrained by structure. Unlike the foundations, which cannot be moved downward by thermal pressure, or the roof, which can be moved upwards relatively freely, the columns and walls that restrain horizontal movement have limited capacities for this undesigned load. There are various mechanisms that in theory restrain the horizontal thermal expansion: (1) shear at the intersecting wall at each plan corner, (2) friction between the wall and the floor structure along the wall length, and (3) resistance provided by the wall columns in bending from load transmitted through the fire-proofing piers. The second and third mechanisms depend on the geometry of construction, since the force transfer from masonry to steel can only take place where the materials are in direct contact. The practice of building the spandrel beams and columns into the wall provides such contact.

Resistance by the columns in bending cannot take place since the distributed load that is created along the columns (up to 200 kN/m) creates moments far larger than the original wind-load design. The presence of intermediate masonry piers at each intermediate column is of no additional help, since those piers cannot resist the lateral load by themselves but will simply transfer it to the same columns. Resistance to the load through friction with the floor structure is resistance through horizontal shear in the walls; analysis of the long north wall shows a required friction/shear stress of 130 kPa on each of the two 200 mm by 30,500 mm contact surfaces per floor, which is realistic under the conditions described. However, on the short west wall, the required friction is 510 kPa, which is greater than the allowable shear strength for most clay masonry, and implies a large compressive force (which may not be present) in order to develop the friction.

Failure of all of the resistance mechanisms results in outward movement of the ends of a wall, cracking the corner masonry on the intersecting walls in line with the inside face of the wall in question. At an ordinary exterior corner, both walls are moving from roughly the same temperature change, and cracks develop on both faces.

5 EFFECTS OF DIFFERENTIAL STIFFNESS

Differences in stiffness between adjacent building elements is one of the most potentially damaging effects that can be found. Modern construction contains numerous provisions for movement to prevent accidental load transfer to relatively stiff elements, including slip joints in curtain-wall mullions, expansion joints in masonry curtain walls, and movement joints in interior partitions. Buildings of the studied type contain continuous masonry curtain walls in direct contact with the structural frames and interior terra-cotta- or gypsum-block partitions solidly built between floor slabs. The relative fragility of the partitions makes them less likely to carry structural load, so the focus here is on the exterior walls.

Similar to thermal effects, the effects of differential stiffness can be examined for vertical and horizontal movement. As thermal effects are limited to the wall plane because the amount of change in wall thickness from temperature variation is negligible, stiffness effects are effectively confined to the wall plane by the magnitude of the element stiffnesses. Masonry walls, subjected to out-of-plane forces are far more flexible than the structural frame to which they are attached and therefore act, properly, as non-structural elements. Walls subjected to in-plane loading have stiffnesses of the same order of magnitude as the frames — often greater than the frames — and therefore have a tendency to carry load. The simplest example is vertical movement under floor loading. As shown in Table 1, the shortening of the frame columns under dead load is greater than the shortening that the adjacent masonry
walls would undergo if they were carrying the load. This means that the load will be distributed between the wall and the frame in proportion to their vertical stiffness until and unless the wall suffers enough damage to release its load. A 300 mm wall would not be designed as the structural support for a 14-story building because the compressive stresses are too high; assuming that the curtain wall was built before all the dead load is in place implies that as the pressure within the wall increased, slip between wall and frame or local crushing of masonry relieved some of the load back to the frame. As live loads increased during and after construction, a similar effect would occur.

The more interesting effect of differential stiffness occurs with sideways from lateral load. The steel frame is strong enough to carry the loads, but is significantly more flexible than the combined frame-and-wall. Wind pressure can move a windward wall (bending in its weak direction) and the frame behind more than it moves the parallel walls. The long north wall is far stiffer by itself than the frame, while the short and fenestrated west wall is more flexible than the bare frame. The walls can only serve to fully stiffen the frames as long as they remain uncracked; each crack reduces the continuity of the walls and therefore their stiffness. This ad hoc composite structure has rarely been discussed. Stockbridge, in his paper on the Woolworth Building, lists a number of mechanisms but looks at all of them in terms of vertical compression within the wall (Stockbridge, 1981). He does not separately discuss the flexure and shear caused within the masonry by sidesway.

Unlike thermal changes, wind loads are highly dependent on conditions immediate adjacent to the building in question. Tall neighbors can block winds, as is true for the north-south wind at 953 Fifth Avenue, and a grid layout such as exists in midtown and upper Manhattan and in the downtown area of Chicago prevents full wind load from being applied to buildings near or below the median height by preventing most winds perpendicular to facades. Exceptional conditions may allow for full wind load: the west face of 953 Fifth faces the 800-meter-wide open space of Central Park. The likelihood of sidesway as the dominant mechanism for cracking can be judged by comparing cracking patterns to directions of possible wind loading.

As Table 1 shows, the absolute difference between frame and wall flexibility is greater for the slender west facade than the stocky north facade. More importantly, the maximum bending stress in the north wall is 97 kPa, acceptable for most masonry, while it is 455 kPa in the west wall under reduced load and 2760 kPa under full code load. In other words, while the masonry contributes greatly to the stiffness of both stocky and slender walls, it is not strong enough to carry the loads its stiffness attracts in the slender walls. This difference applies in most buildings of this type since, in the era before air-conditioning and fluorescent lights, buildings with large overall plan dimensions had light courts that create multiple slender wings (Fig. 4).

6 EFFECTS OF RUST-JACKING

Unlike intermittent wind pressures and thermal expansion, pressures caused by rusting steel are continuous and (until repairs are made) ever-worsening. The pressure increases from zero as rust builds up behind the masonry, forcing it outward. The volume of rust compared to base metal varies with, for example, Gibbs providing values of seven to twelve times the original volume (Gibbs, 2000). These large values represent free expansion. When masonry is solidly built against a rusting piece of steel, the volume increase is restrained by the strength of the masonry. In technical terms, rust-jacking is the deflection of that masonry under load until failure (typically through cracking) and the forced movement of the masonry after failure, including bulging, spalling, and collapse.

In buildings of this type, the structural steel was protected with red-lead paint and, by virtue of being embedded within a masonry wall, has never been subjected to mechanical abrasion, impact, or exposure to attack by any non-water-soluble chemical agents. Experience has shown that the combination of paint and masonry skin performs well in flat and ordinary sections of wall. Where a greater than normal source of water entry into the masonry exists, such as parapets and the top surfaces of applied ornament including cornices and water-tables, damage is likely...
to occur. Rust-jacking, in other words, is the structural equivalent of an opportunistic infection. While the masonry and paint that provide weathering protection to spandrel beams and columns fail over time, the most serious rusting occurs in those areas where the masonry fails first, from poor detailing that aids water entry, improper construction, or most often, cracks from thermal and lateral movements. This secondary effect is the most serious form of damage in high-rise buildings since it is the only one that is progressive and reduces structural capacity: if rusting of structural steel were not a concern, it might be possible to accept many of the cracks formed by movement as "naturally-occurring" expansion joints that posed no real danger to either people or property.

Because the effects of rust-jacking are non-linear — the restraint provided by masonry drops from full to zero after cracking — and are highly sensitive to voids in the column fire-proofing piers and in the veneer outboard of the spandrel beams, it is not possible to analyze this effect directly.

7 COMMON REPAIR TECHNIQUES

Since the passage of Local Law 10 in 1980, the New York City Department of Buildings has required inspection of all building facades higher than six stories every five years (Local Law 10, 1980). Conditions identified as dangerous must be repaired shortly after the inspection report is filed (Prior to the 1998 revision of the law, a category of "precautionary conditions" existed between "safe" and "dangerous;" conditions so noted had to be repaired before the next inspection.). A large industry has developed among architects, engineers, riggers, and masons in performing repairs. Given the physical similarity of buildings in the type under consideration, similar weathering exposure, a general lack of maintenance beyond ordinary joint pointing, and the small community of professionals and contractors involved in "Local Law 10 repairs," standard methods of addressing problems developed quickly in the 1980s.

The most common repair performed is rebuilding masonry at external corners. Cracks on both wall faces are common, highly visible, and suggest a section of loose masonry. In the majority of buildings that have brick or terra-cotta veneer, the corner masonry units are typically removed, the steel painted, waterproofed, and repaired as required, and a new masonry corner constructed. Similar repairs are made at horizontal strips over spandrel beams where the beams have rusted (Figs 5, 6). Expansion joints may be created in conjunction with masonry replacement or by themselves. The removal of corner masonry provides an opportunity to create expansion joints at both faces of the corner with minimal effort and the removal of masonry outboard of a spandrel beam provides a similar opportunity to create horizontal expansion joints below the level of wall support.

It may seem obvious to state that any steel exposed by the removal of masonry should be painted and waterproofed to prevent future deterioration, but this work is not always performed. It is not uncommon for workers to open up a ten-year-old repair to masonry and find unpainted, rusting steel. The use of self-adhering rubberized-asphalt sheet waterproofing has
made protecting complex steel shapes from water relatively simple as long as there is enough of a void between the steel and masonry at complex corners (such as the intersection of spandrel beams and corner columns) to allow for some movement of the sheets.

Steel repairs may be performed with masonry repairs, ranging from reinforcing plates welded to flanges and webs that have lost material to rust up to complete replacement of columns and beams. Repair is favored over replacement as it is far less disruptive to the occupied interiors and more safely performed from scaffolding over occupied sidewalks.

8 CONCLUSIONS

Regardless of the causes, damage to masonry curtain walls on high-rise buildings must be repaired to maintain public safety. However, different causes of damage may require different repair details. At this time, despite the large and growing body of experience in New York among designers and contractors with repairs, analysis is rarely performed to discover the causes of observed damage. The mechanism of rust-jacking is well understood, and the initial cracks are typically described as "thermal movement." Two aspects of thermal expansion are not obvious: first, that the force developed in restrained thermal expansion is independent of the length of the element; and second, that simple friction with the floor structure is, for long walls, more of a restraint on thermal expansion than that provided by intersecting walls and columns. The result for this type of construction, counter-intuitive for non-engineers, is that the longer a wall is, the less force it exerts from thermal change at corner intersections with other walls.

Lateral load is rarely considered in damage surveys of these buildings, since the steel frames are typically adequate for all lateral loads and current design practice does not typically treat steel and masonry as a composite structure. Analyzing this effect can give an upper bound on stresses in the masonry and, more importantly, can provide insight into patterns of facade damage, and therefore into repairs required. For example, if thermal expansion is believed to be an important factor, vertical expansion joints may be cut into large flat wall planes as well as at corners, while if sideways is actually the dominant factor, the corner joints are all that is required.

The analysis and repairs techniques that have been developed in New York (and similar techniques in Chicago, since the 1996 enactment of a façade investigation law) are mostly empirical. The advantage to this approach is that only those techniques that produce acceptable results become commonly used. The disadvantage is that a lack of explicit understanding of a given problem leads to an iterative repair approach, which is wasteful of time and money. Analysis of facades as composite steel-and-masonry structures, as suggested here, is one method towards better focused repairs.

REFERENCES

1980 Local law 10/80 in The City Record 108(32169).