



A CASE STUDY IN STRUCTURAL RENOVATION AND MASONRY RESTORATION

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ABSTRACT

A four-story unreinforced brick masonry building in Cleveland, Ohio was studied for the purpose of performing structural renovation and restoration of the brick, terra cotta and limestone masonry façade. New Code provisions from International Building Code 2000 (IBC) and International Existing Building Code 2003 (IEBC) were applied. A reinforced concrete addition to the building was designed to carry the bulk of the seismic load. An opinion of cost was developed for the masonry façade restoration.

KEYWORDS: renovation, restoration, evaluation, clay tile, remediation.

INTRODUCTION

The four-story South Building at Case Western Reserve University (CWRU), a prominent school located in the eastern suburbs of Cleveland, Ohio, USA sits on a 67,760 m² (14 acre) site that contained approximately a dozen buildings. The entire site comprised the former Mount Sinai Hospital before CWRU purchased the property in the early 1990s for expansion of the campus offices and laboratories.

The South Building was originally built in 1914 as a collection of wings that jutted out from a central stem running in the east/west direction. After an extensive feasibility study of all the buildings on the property, the easternmost wing of the South Building was identified for rehabilitation, while the remaining portions of the building were to be demolished. This portion of the building was selected to remain largely because of the classic architecture of its east façade (see Figure 1) and for its existing floor live load capacities.



Figure 1 – Existing east elevation of the 1914 South Building

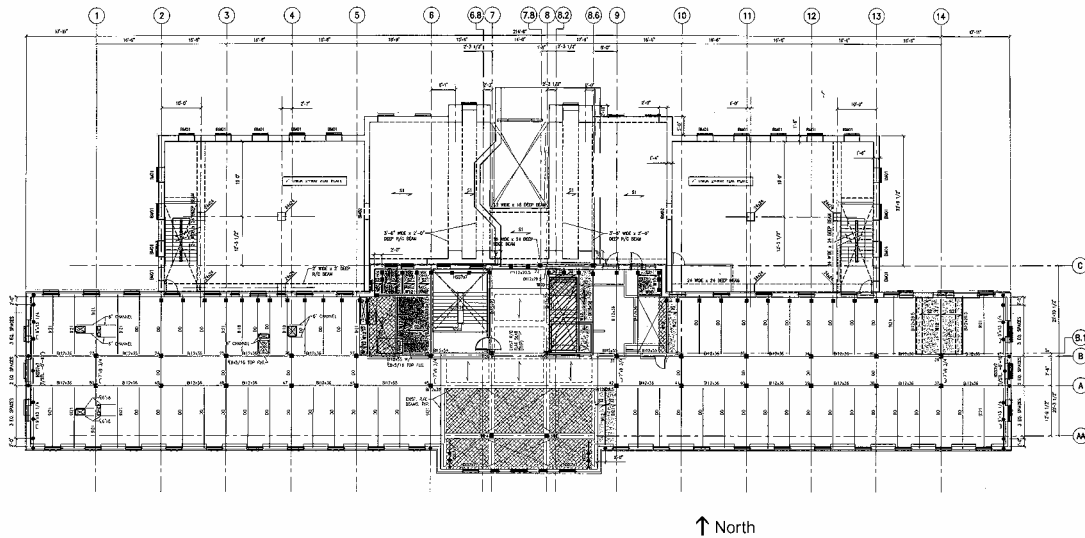


Figure 2 – Typical South Building floor plan. (In general the original building is south of Column line C, and the new portion is north of line C)

Thornton-Tomasetti Group, Inc. of Chicago, Illinois, USA provided the structural engineering services for the feasibility study, demolition, and rehabilitation projects for the South Building. Making this project all the more challenging was the lack of complete original drawings or other documentation. While some drawings were found, many locations within the building did not match the drawing geometry, and no record was found of original material design values.

BUILDING DESCRIPTION – INFORMATION GATHERING

Once the interior finishes of the building were removed, an exhaustive structural survey and material testing program was conducted. The survey revealed that the floor framing consisted of a 250 mm (10 in) deep flat clay tile arch floor system, an archaic structural configuration that utilizes clay tile cut to form compression arching action (see Figure 2). This clay tile was covered by a 25 mm (2 in) thick cinder fill topping. The tiles are tied with an embedded tie rod at mid-depth that resists the thrust of the arch through tension action. The tie rods are anchored to steel beams spaced at 1.7 m (5.5 ft) on center. The 225 mm (9 in) deep steel beams frame into 300 mm (12 in) deep steel frames and girders. The floor steel eventually spans to either interior steel columns or the exterior brick bearing wall. The exterior bearing wall thickness consists of three unreinforced brick wythes (roughly 300 mm (12 in) total in thickness) that include a header row every six vertical courses.

Material design strength information was gathered on the exterior wall and steel columns through a process of material testing. The steel framing was researched and determined to have a yield strength equal to an average of approximately 190 MPa (27.5 ksi). The exterior load bearing brick was also tested for shear using a Coulomb in-place shear test (see Figure 3). This test was performed in accordance with guidelines specified within the IEBEC A106.3 guidelines. The in-situ mortar shear design value, v_t , was determined to be an average of approximately 1.39 MPa (201.5 psi), a surprisingly high test result for a ninety year old exterior brick wall that has been continuously exposed to the elements. The testing was performed by PSI, a professional engineering testing agency in Cleveland.

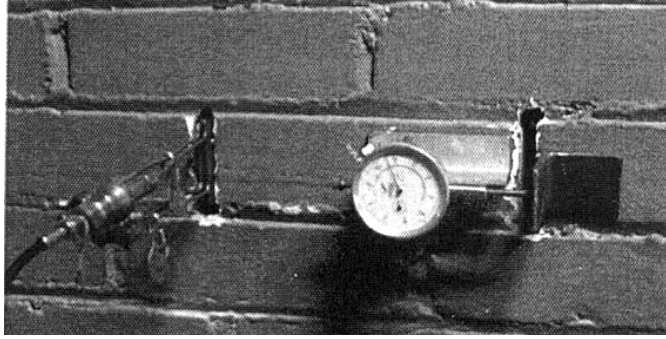


Figure 3 – Coulomb in-place shear test according to IEBC A106.3

STRUCTURAL EVALUATION

The building structural evaluation was performed in three main areas: the gravity system, the foundation system, and the lateral system.

The proposed gravity live load of 3,830 Pa (80 psf) for new office loads [2,395 Pa (50 psf) live load plus 1,435 Pa (30 psf) movable partition load] was relatively easy to verify. First, the original documents indicated an allowable live load of 3,830 Pa (80 psf) for the floors. These values were independently verified by surveying the floors, and reviewing flat arch design tables from the textbook entitled “Hollow Tile and Fireproofing,” copyrighted in 1919, and the 1924 American Institute of Steel Construction (AISC) Standard Specification. The steel floor framing for the most part was adequate with minor exceptions. The biggest concern for the clay tile flooring was localized damage typically on the underside of the floor due to interior construction of ceilings, lights, and similar systems. On some levels this accounted for up to 10% of the total floor area.

While most individual floor loads did not increase, some change in use requirements, and heavier roof loads, caused an approximate 20% column load increase from the original design to the proposed program by the design team. The column analysis was performed with formulas published by the AISC historical record of Dimensions, Properties, and Rolled Shapes compiled by Herbert Ferris, and using the book Historical Building Construction by Donald Friedman, 1995. A check on the columns for the new loads determined that only approximately 15% of them needed to be reinforced for the additional compression stresses. This reinforcement was primarily due to the addition of mechanical units to the roof of the building as part of the necessary renovations.

The foundation system underwent a fairly extensive investigation by the Geotechnical Engineer that included several test pits to verify the geometry and depth of the footings. The original design documents noted an allowable bearing pressure of 95,760 Pa (2,000 psf). However, the as-designed renovation program would impart up to 287,280 Pa (6,000 psf) of bearing pressure on some of the footings. After additional in-situ pressure meter and other tests were performed by the Geotechnical Engineer, we concluded that the existing concrete foundations were adequate to resist the program gravity loads. We believe that the higher soil capacity was realized as a result of an underestimation of the original soil bearing capacity coupled with subsequent consolidation of the soil underneath the existing foundations over the last 80 years.

The lateral force resisting system study required extensive investigation into the exterior brick bearing wall as well as the existing structural steel frames. Early analytical models of the partially-restrained steel moment frames, under wind loads alone, indicated insufficient strength and stability. Therefore, we came to the conclusion that the extensive existing unreinforced brick masonry walls provided the bulk of the lateral force resisting system for the original 1914 building.

Conversely, the minimum design requirements for walls, especially those that form a part of the lateral system, have increased. The jurisdictional Code adopted by the City of Cleveland and in effect when the project was under study and design in 2003 was the IBC. This relatively new Code required assessing the structure for both wind loads and seismic loads. At the time of construction, no seismic design criteria were required within the City of Cleveland. We anticipated that the existing building would, at a minimum, need a significant retrofit to the lateral system in order to satisfy the current Code requirements.

The seismic design requirements within IBC are based on a response spectrum model derived from the Newmark-Hall method and considers a seismic event having a 2% chance of occurring in 50 years (FEMA 303, 1997). This response spectrum model also assumes 5% damping. The response spectrum unique for this building is created by assembling the peak ground accelerations (S_1 and S_s) and the site soil condition, both of which are unique to the site. The accelerations are obtained from hazard maps assembled by the U.S. Geological Survey, while the site soil condition is given from the Geotechnical Engineer's study of the site conditions. To complete the response spectrum, the Seismic Design Category is determined from the IBC code tabulated values within section 1616.3. A summary of the dynamic characteristics for the South Building is prepared below:

| | | |
|-------------------------|---|--|
| S_s | = | 0.215g (short period spectral response acceleration) |
| S_1 | = | 0.058g (spectral acceleration for a 1-second period) |
| Soil Site Class | = | C (Very Dense Soil) |
| Seismic Design Category | = | B |

The first step in the structural evaluation of the lateral system was to compare the lateral loads from the original building design in 1914 with the Code required loads within the City of Cleveland at present day (refer to Figure 4 below). The lateral analysis procedure is described later in this paper and was constructed within ETABS[®]. The results show that the minimum required demand that the 1914 South Building must resist has increased by 400% due to new Code requirements and updated information kept by the U.S. Geological Survey for the geology within the State of Ohio. This is much larger than the 5% overstress limit permitted for renovated structures by IBC Section 1614. It was therefore determined that retrofit of the existing system would be untenable, and a new lateral load path must be developed in order to renovate this building.

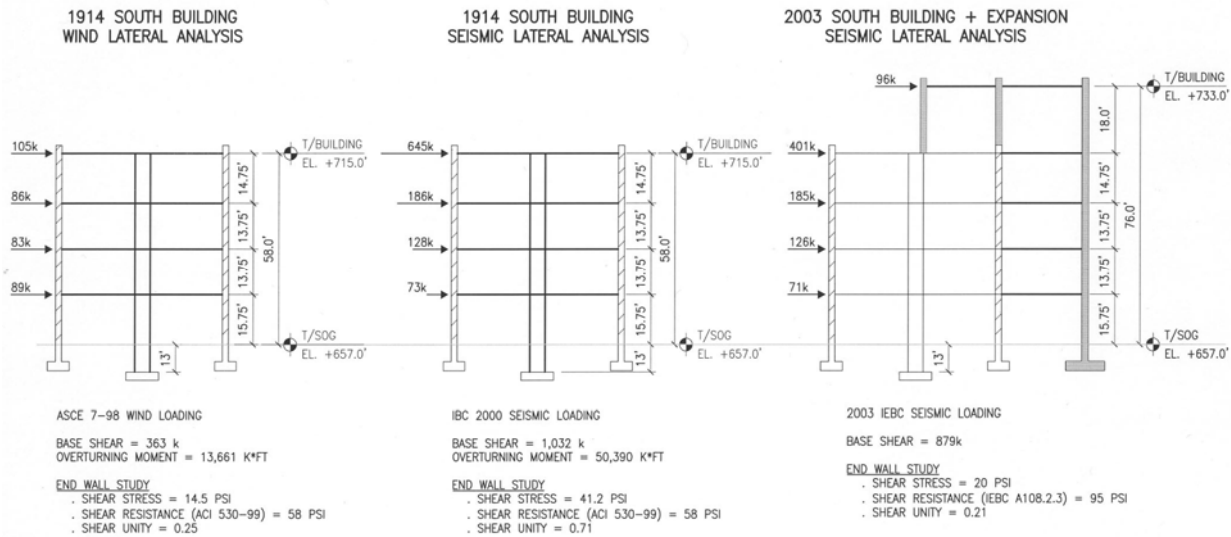


Figure 4 – Lateral Load Analysis Summary on the portions of the 1914 South Building.

STRUCTURAL REMEDIATION-GOALS

In order to provide a successful remediation to the remaining East Wing of the South Building, a new lateral load path needed to be constructed. The new lateral force resisting system needed to meet four main criteria or goals:

1. The new lateral system must be sufficiently stiff to prevent the existing exterior brick wall from undergoing large deformations.
2. The new lateral system must be sufficiently strong to successfully resist the high forces generated by new seismic design criteria.
3. The new lateral system must be sufficiently economic such that the total cost of renovation would be less than 75% of the cost of a new building of equal square footage.
4. The new lateral system must not take away from the classical architecture for which the existing South Building is known.

STRUCTURAL REMEDIATION-RECOMMENDATIONS

Many different lateral systems and configurations were studied. Inserting steel bracing into the existing building would segregate the interior space into cells that would make the space impractical for a modern office. In addition, the bracing connections would require large, unsightly through-bolt connections into the exterior wall, which did not meet goal number 4 listed above. Inserting concrete walls around the internal stair and elevator towers would require extensive renovation of the foundations. The shear walls and supporting foundation work would be performed with space restrictions and proved to be too costly to meet goal number 3.

CWRU selected an option which added 2,790 m² (30,000 square feet) to the building along the West side. This addition would add seven reinforced concrete shear walls and would tie into the existing building at each floor level. The proposed addition would serve as the main lateral system for itself and the remaining portion of the South Building. Best of all, the added square footage enabled the cost of the renovation to meet goal number 3 (less than 75% of the cost of a new building of equal square footage).

The final recommendation made to CWRU was to consider using the IEBC 2003 as the primary analysis guide for the remediation of this building. The benefits of using this Code are many for existing buildings. In addition, this Code contains an entire section entitled “Seismic Strengthening Provisions for Unreinforced Masonry Bearing Wall Buildings” within Appendix A, Chapter 1. This chapter provides guidelines which relax the strength criteria for unreinforced masonry required for new buildings based upon the past performance of historic buildings of similar structural systems.

The IEBC 2003 had not yet been adopted by the City of Cleveland at the time that the design drawings were being implemented. Therefore, the engineer had to present to the owner and architect the value of the document, present the information to the Cleveland Department of Buildings for the purpose of obtaining a variance, and provide background information to the University’s insurance company for acceptance of the design level presented within the code. All three groups reviewed the document and approved it for implementation on this building.

STRUCTURAL REMEDIATION-BASE SHEAR CALCULATIONS

The creation of a new lateral system was one of the major financial hurdles for this project. In order to understand the solution proposed above (to add onto the building in order to resist the updated seismic code demands), a brief review of the lateral analysis in the form of base shear comparison is given below.

Within the IBC the computed seismic base shear V is given as

$$V = C_s W \quad \text{IBC Equation 16-34}$$

$$\text{where } C_s = \frac{S_{ds}}{\frac{R}{I_e}} \quad \text{IBC Equation 16-35}$$

$$\text{or } V = \frac{S_{ds}}{\frac{R}{I_e}} W \quad \text{IBC Equation 16-34}$$

C_s is calculated to be a percentage of the total seismic dead load (W) of the building. The seismic dead load (W) includes the dead load of the building plus a minimum of 480 Pa (10 psf) for partitions, 25% of storage area live loads, heavy equipment operating weights, and 20% of the roof snow load. The C_s term attempts to model a percentage of the seismic dead load (W) that an earthquake will convert to a horizontal inertial force once the building begins to sway. C_s also is a function of the natural frequency of the building.

The C_s value computed for the South Building is 0.143, yielding a seismic base shear equivalent to 14.3% of the seismic dead load. The weight of the existing South Building itself is approximately 50,285 kN (11,300 kips). The weight of the existing South Building plus the proposed addition is approximately 70,310 kN (15,800 kips). Proportioning the base shear to the effective stiffnesses of the existing building and the addition, the proposed addition would resist approximately 70% of the total shear.

The allowable drift of the structure is most dependent upon the R value chosen for IBC equation 16-34. The R value primarily models the ductility of the structure and, for the new reinforced concrete wall system, is prescribed by Code to equal 3.0. The R value for the existing exterior unreinforced brick masonry bearing wall is 1.5. Because 30% of the total lateral seismic force will be resisted by the existing unreinforced brick masonry bearing walls, these walls may not sway or deflect an amount that may impact their ability to support the floors. Therefore it is prudent, as well as required by Code, to use R=1.5 for the entire combined structure and minimize any horizontal movement the building may experience in a seismic event.

The smaller R value creates a larger anticipated seismic force. However, the IEBC allows a reduction of the seismic forces for existing buildings. The benefit of using the IEBC is that the computed seismic base shear used the $\frac{3}{4}$ (0.75) factor as permitted by IEBC Appendix A, Eq. 1.7. This 0.75 factor is only allowed to be applied toward the inertial forces generated by the existing building, while the full inertial effects of the seismic force are generated by the new addition.

STRUCTURAL REMEDIATION-EXISTING UNREINFORCED MASONRY WALL CALCULATIONS

As mentioned above, the exterior unreinforced masonry wall was determined to be the original primary lateral resisting system for the 1914 building. This wall is relatively brittle, which makes it susceptible to damage from load reversals or a sudden large overturning moment. It is important to note that the walls were constructed at a time when reinforcing bars were not commonly used with brick masonry construction.

In addition to having no reinforcement, the exterior walls have numerous window penetrations. Consequently, at the window line the exterior wall is broken up into a series of piers. The piers collectively must take the same overturning moment as the entire wall, but individually have a very small moment arm to resist overturning forces. After performing a lateral analysis, it was found that net tension did occur from in-plane seismic forces at the wall piers despite the addition of seven new reinforced concrete shear walls. The IBC does not allow in-plane tension within unreinforced walls. One option to renovate the walls by coring into the walls and adding steel is extremely expensive. Another option, to add fiber reinforced polymers (FRP) to both faces, can be expensive and require labor specialties. The IEBC, however, does allow a minimum tension for in-plane walls. By using the IEBC procedures for evaluating in-plane shear walls, the material tested values for shear in the brick masonry according to IEBC Appendix A, Section 106.3.3, and the additional reinforced concrete shear walls, TTG was able to prove the existing walls were adequate to resist the updated seismic forces.

There were some cases where the existing exterior wall penetrations were too large or too frequent. This occurred at locations near the corners of the building or where there had been doorways. In these cases a detail was provided that would in-fill the existing wall in order to prevent tension forces from exceeding IEBC values.

The final study performed on the existing brick masonry walls analyzed the out-of-plane demand created by the updated seismic load requirements. Upon initial review, the tension forces created out-of-plane exceeded the allowable capacity prescribed by the IBC. However, the IEBC does address specific conditions for the re-use of antiquated unreinforced masonry buildings. IEBC allows existing unreinforced masonry walls to be examined by a maximum wall height-to-thickness ratio as detailed in IEBC Table 1-B, which were not exceeded for the South Building renovation design.

STRUCTURAL REMEDIATION-DIAPHRAGM COMPATIBILITY

The archaic clay tile arch floor presented a number of challenges. This floor system is just as brittle as the existing unreinforced brick walls. The floor system must be stiff enough to transfer seismic forces to the lateral resisting system. The floor system must also be strong enough to take in-plane bending moment due to its own inertia. To prevent tension forces from building up via in-plane bending, new concrete shear walls were strategically placed to ensure the in-plane diaphragm did not span farther than 5.18 m (17 ft), thus minimizing the amount of tensile forces that could build up from in-plane bending. This kept the diaphragm stresses low enough that it did not require any strengthening for seismic forces.

We had additional concerns about the clay tile arch floor system and how it connects to the exterior wall and the new addition. IEBC Appendix A, Section 113.1 requires that a positive floor-to-wall anchorage must occur at a maximum spacing of 1.83 m (6 ft). Exploratory excavation of the beam pockets showed that a 250-300 mm (10 – 12 in) long pin was cast into the brick wall, and drilled through the center of the steel beam webs. Since the beams were spaced at 1.68 m (5.5 ft) on center, no additional anchors were required to the existing wall. The one exception to this was at the corners of the building where IEBC requires anchors within 1.22 m (4 ft) of the interior face of each corner.

A fair amount of work was performed in developing the ties from the existing building diaphragm to the new addition diaphragm. The ties were evaluated for actual strength transfer required to resist the diaphragm forces generated, the out of plane bending forces created by the unreinforced masonry wall, and according to the IBC 1620.1.4 minimum connection force. These anchor details were required to transfer over 60% of the existing building seismic load into the new addition.

STRUCTURAL REMEDIATION-CLAY TILE FLOOR REPAIRS

As mentioned above, the clay tile floor system plays a critical role in the transfer of diaphragm forces from the existing building to the new addition where new shear walls can adequately resist updated seismic forces. During the investigation of the original 1914 building, it was found that up to 10% of the clay tile floor area contained damage. This damage ranged from flange holes the size of a finger to full-depth holes 4 feet in diameter. The purpose of the repairs was only to retrofit the clay tile well enough to restore a continuous surface that would be level for pedestrians and strong enough to resist the punching shear demands for most office equipment. As mentioned above, the additional concrete shear walls and their location kept the diaphragm forces below the existing demand already being experienced by the diaphragm for shear and tension.

A criterion was developed for identifying holes that required repair as well as appropriate repair details for full depth repairs and partial depth repairs. A survey of all the damage was required, and any bay with six square feet or more of clay tile bottom flange damage was to be removed and replaced. After studying a number of different damage patterns, an area of more than six square feet was determined to require that the tile be removed and replaced. Repair to smaller areas of localized damage were performed by providing a bulkhead of foam around the damage and filling the clay tile cell with a lightweight, durable concrete filler. This enabled the clay tiles to be repaired without adding large amounts of dead load to the structure.

STRUCTURAL REMEDIATION-FAÇADE RESTORATION

In the summer of 2003, an investigation of the façade repairs that would be necessary for a successful renovation of the South Building was initiated. At this point in the project, only the east, or front, elevation was studied, as this appeared to be the only elevation that would remain largely unaltered after renovation. A preliminary budget for repairs on this elevation had been submitted to the construction manager by a local specialty contractor and totaled approximately \$400,000.

A particularly interesting detail that was identified early in this process involved the structural steel elements that supported the terra cotta cornice near the top of the building. The cornice already had an infamous history—an 890 N (200 lb) piece of it was found lodged in the roofing of the nearby, lower building roof. This piece was later identified as having fallen from the southwest corner of the South Building cornice (see Figure 5).



Figure 5 – South Building cornice, southwest corner

The construction manager's field personnel pointed out to us one day that a significant horizontal crack within the brick masonry, noticed only from inside the building, occurred below each floor. Upon further field study, the horizontal crack was revealed to be a large joint separation, only at the uppermost floor of the building, resulting from corrosion of an embedded steel angle that ran continuously along the length of each brick wall. Beneath this angle we discovered the ends of structural WT members, spaced at roughly 600 mm (2 ft) on center. The angle running perpendicular to and on top of the WTs did not appear to be connected in any way to the WT elements.

Without removing any more of the cornice, we were able to determine that the WTs provided support for the dentil work observed within the cornice construction, and that the angle on top of the WTs may have been added during construction simply to tie down the cantilevered steelwork. We also determined that significant corrosion of this angle had pushed the parapet wall upward and would require full rebuilding of this upper part of the masonry wall in order to achieve structural stability as well as weather tight construction.

Replacing the terra cotta cornice and rebuilding the parapet wall were two items that were glaringly absent from the preliminary budget prepared by the local specialty contractor; our engineering opinion of cost for the East elevation was consequently much higher, around \$725,000 for the repairs to the façade elements.

SUMMARY

The renovation of a 1914 unreinforced brick masonry bearing wall building was examined in consideration of the new IBC and IEBC Model Building Codes. A reinforced concrete addition was designed to both add square footage to the project as well as provide sufficient lateral force stiffness and resistance for the Code wind and seismic loads. The combined system was analyzed as a non-ductile ($R=1.5$) structure where the new reinforced concrete walls of the addition would take 70% of the lateral load, and the existing brick masonry and clay tile diaphragm system would limit the lateral drift permissible in the design. By adding the additional lateral system with usable square footage, the existing portions of the building did not require retrofit. This idea enabled the cost of the entire project to be very near the target of 75% of the cost of a new building of equal square footage.

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