PRESQU'ILE POINT LIGHTHOUSE (PPL): RESTORATION ENGINEERING STUDY

Prepared by:

André Scheinman
Heritage Preservation Consultant

John Silburn
Restoration Engineering

November 14, 2014.

PRESQU'ILE POINT LIGHTHOUSE (PPL):
RESTORATION ENGINEERING STUDY

TABLE OF CONTENTS

PRESQU'ILE POINT LIGHTHOUSE (PPL):
RESTORATION ENGINEERING STUDY

Introduction

The Presqu'ile Point Lighthouse is an important heritage site located at the tip of the Presqu'ile peninsula (originally Gibson's Point) extending into Lake Ontario within Presqu'ile Point Provincial Park, just west of Brighton (Fig.2). It is one of the earliest of Ontario's surviving lighthouses and is an essential part of the history of shipping on Lake Ontario and the Great Lakes in general. Constructed to the ambitious design of engineer Nicol Hugh Baird, the almost 70' high octagonal structure was built in 1837-1841 in stone and featured tall pointed arch windows, the hallmark of Gothic Revival architecture (Fig.1). Unfortunately, almost immediately upon completion leakage at the metal roof and lantern into the stonework was noted and, despite attempts at remediation, a spiral of structural degradation set in motion which culminated in the decision by the government in 1894 to stabilize the structure with iron bands and timber framing while protecting the stone from further weathering by cladding in wood shingles. This strategy has been essentially successful and, despite having its lantern removed (1966) and no longer functioning as a working Lighthouse (though still a navigational aid with skeleton tower), the building survives. Along with the stone Lighthouse Keeper's House/Interpretive Center (1846, restored and incorporated into an Interpretive Center), the Lighthouse forms an important component of Presqu'ile Provincial Park and the nautical history of the Province. The Friends of the Lighthouse (KOPPLA – Keepers of the PPL Association) in association with Ontario Parks have initiated the current study – which will examine the existing condition of the Lighthouse and, from that knowledge base, make recommendations with regard to its preservation - including options for its restoration including consideration of the feasibility of restoring a lantern to surmount the stone structure.

Fig.1: Baird sketches for PPL

Fig.2: Presqu'ile Peninsula (Note: North is 'left')

Methodology

A two-fold approach has been taken to gaining the best understanding of the Lighthouse . One aspect has been the careful review of background documents, particularly: the original specifications; the early reports expressing concerns by Inspectors of the Marine and Fisheries Department (including the decision to gird and clad the building); later maintenance records; the 1983 Lighthouse Feasibility Report and the 1993 Park Master Plan.

Fig.3: PPL Front (West) elevation

The other aspect has been the undertaking of a relatively comprehensive on site investigation of the structure's existing condition. This has included: detailed examination from the interior – including predrilling the masonry and probing with a borescope in an attempt to examine the core of the wall; an examination of the existing metal roofing from the roof itself; exterior inspection with binoculars but also close-up from a 100' man-lift. The latter allowed the removal, in strategic locations (south and southeast elevations), of shingles and wood framing to provide 'windows' through the exterior cladding/framing to view the exterior stone and to, again, drill for borescope penetration – thus viewing areas of the core not reachable from the interior. The borescope probes were able to be viewed in 'real time' on a laptop but also were recorded for later reference. In all - five locations were probed at the interior and three at the exterior. Note that the extent of framing (wood girt) removal was limited due to concern for creating any point of weakness in this well established stabilization system.

The information gleaned from this approach has now been analyzed leading to the discussion and recommendations below.

Chronology of Structural Issues

As noted earlier the 69 ' high stone structure, rising on a 'batter' from a 30' square base as an octagonal with 6' thick walls at the bottom and 2'-6" thick walls at the top (with a relatively consistent interior ‘well’ of almost 12’ throughout) was plagued with leakage problems from the outset. (See Sketches 4-5). Quoted (and/or paraphrased) below are some of the salient points from the original specification and subsequent key observations and actions undertaken based on inspections of the building over the last 170 years. The notes/comments in *italics* are the authors.

1837: N.H. Baird, P.Eng. developes original contract docs for erection of PPL

> “Quoins are to be ashlar 15” in thickness min. One or two courses of hammer dressed stone is to be in bond with each quoin… Courses to consist of a header stone for every two stretchers….the hearting of the wall to be laid flush in well made mortar till it oozes out and each course well packed and grated(?) before the next commenced.
>
> The corners and blocking(?) courses were to be laid in “water lime”; the foundation with “hot lime and cement mixed.” If “iron shine dust” could be procured the “water lime” could be dispensed with.

Of interest is the extent of header stones spec’d. These were to be “not less in depth into the wall than 3x their own thickness”. A*lso there is reference to “water lime” - this would seem to refer to lime with hydraulic qualities. Then there is the mention of* “iron shine dust” - *obviously considered a desirable additive as a pozzalan. Of concern is the description of the* “hearting” *of the wall, i.e. the core, which is simply to be filled with mortar (and tamped) as each course is brought up. This seems to describe a system which, with its potential lack of through wall bonding and reliance on mortar fill of the core would be susceptible to the separation of wythes and settlement of the core in the face of moisture and repeated freeze-thaw cycling.*

1842: G.H. Dunlop Inspection Report

> *Water seems to have been pouring into the Lantern interior and the stone copings from poorly executed sheet metal joints:*
>
> **“**Water has penetrated every coping joint. Stuff used for pointing was useless.”
>
> “3rd floor wall plaster beginning to scale off.”
>
> “ Base courses need repointing and parging.”

1842: Baird developes a spec to remediate these problems particularly as relates to the Lantern and the roofing

1894: Inspection Report of Marine and Fisheries

"…due to bad stone and workmanship …tower has cracked badly and become unsafe." …To be repaired by surrounding with iron bands and planking and shingling the sides."

1895: Inspection Report notes above work completed.

1966: Lantern removed and replaced with skeleton tower.

Government of Canada - Supervisor of Marine Operations

1970's: Shingles at <u>bottom</u> section noted as requiring replacement every few years

1981: Windows replaced with steel plate (largely due to vandalism). Possibility of turning Lighthouse over to Provincial Park.

1982: E.A. Cromarty Architect and Roney Engineering: Lighthouse Feasibility Report:

Opened 'windows' through cladding which was found to be:

" …girded with horizontal 2" x 4"s on edge tightly spaced near the base and spreading to 4" centers at the top. A cladding of cedar shingles is applied over the girts."

On windward sides mortar reduced to sand for depth of 12". Likely that exterior wythe would have to be dismantled and rebuilt/replaced if stone were to be left exposed.

Timber lintels badly decayed.

Except for some loose stones at NW corner **the foundations are sound and in good condition."** *Note: A test hole was done at that location.*

1993: Chris Borgal Architect as part of Park Master Plan Report:

Opened 'windows' though cladding (see attached sketch of cladding/girting section) though only at lower portion of the building.

Timber support structure found generally to be in good condition though an area of potential frass noted.

Shingles at south and southwest had outlived service life.

None of the iron straps found.

Mortar joints of varying size but generally quite wide. Postulates that extent of sand in basement indicates volume washed out of joints and core.

Postulates that stone is inferior local shore stone. Asserts that spec called for Kingston stone though this reference actually is in the spec for the Lighthouse Keeper's Residence. Supposition that stone was laid green rather than weathered/hardened.

States that mortar is 1 lime to 3 sand "as specified" but no reference to this ratio, though certainly typical for the time, in Baird's spec. Rather Baird talks about 'water limes' and cement for certain critical areas. This mortar formula is referred to however in the specs for the Lighthouse Keeper's Residence.

Notes that window sill detail would allow water to run into the stone work at back of wood sill when interface joint deteriorated.

Viewed aggregate under micro and asserts that it is rounded beach sand rather than the 'sharp, angular sand' ideal for stonework and spec'd by Baird.

Major relevant conclusion is that the timber banding is essential to maintain the stability of the structure <u>"as the tower itself cannot be made structurally independent of the banding."</u>

<u>Summary:</u>

Leakage was occurring into and through the stone walling from the time the building was completed in 1841 until the completion of stabilization/weatherproofing measures in 1895. Initially leakage appears to have been from the top down but eventually through deteriorated, wide mortar joints. The nature of the construction (' mortar filled hearting' at core of wall), wide mortar joints filled with questionable mortar, and likely, lack of enough through wall bonding, combined with a severely weathering site seems to have led to failure in the form of major cracking and, possibly, the almost complete disengagement of the outer wythe of stone, at least at the south and south west elevations.

However, the foundation was considered stable by Roney Engineering in 1982 and both studies conclude that the 1895 stabilization/weatherproofing approach has been successful. A major disparity between the 1982 and 1993 engineering findings was that the 1982 study. While acknowledging the seriousness of the masonry problems still felt that it would be possible to expose the stone while the 1993 Study did not feel that such an approach should even be considered.

<u>Existing Condition</u> (See Sketches 1- 3)

<u>Roofing</u>

The existing roofing is a low slope batten seam treatment in galvanized metal with the 1 ¼" wide battens typically set at 29" o.c. It is assumed that the seams are soldered with the battens allowing for some measure of movement. The treatment at the roof edge is unusual as the drip edge piece appears to have been applied over the roofing putting that seam in the path of water flow (Fig**.** 4-5).

Fig.4

Paint is flaking over much of the surface and has been completely lost over approx. 25% of the surface area much of which is now corroding, particularly adjacent to the battens. Holes were noted in the metal which have the appearance of being caused by the attempt to remove ice with a pick or axe. Several occur at the metal 'fascia'. (Fig. **IMG 0116)** The area around the pipe-type guard rail's steel base plates, which seem to be simply bolted through the roofing and caulked around the perimeter are a typical problem area. Several bolts are missing and the caulking is basically coming away from around the plate edges making the assembly vulnerable to leakage.

The amalgam of potential leakage points noted above constitutes a major moisture entry problem.


Fig.5: Note hole at edge

Interior Plaster

Plaster was originally applied directly to the stone and white (lime) washed on a continual basis for hygiene. It appears that in association with the stabilization measures undertaken to the Lighthouse in 1894 the upper (Fifth)storey was furred out, lathed and plastered. Certainly the sawn lath, cut nails etc. appear to be from that period. Due to the wall being built out a wood base with beaded top edge was also installed. It is possible that the decision to treat the wall in this way at this floor was necessitated by the level of damage to the original plaster/stone due to moisture and/or modifications at the top of the stonewall.

This later plaster is missing at a number of locations and the exposed lath and furring were found to be decayed in places and lathing nails corroding. On the day of this visit (August 7, 2014) these wood elements were quite damp presumably due to the leakage noted above and/or condensation occurring at the cool stone surface. It appeared that moisture was particularly high in the space between the inner and outer plaster surfaces. (Fig.6)

At the floors below, the original plaster (with its many coatings of lime wash), where not impacted by later interventions (such as around the windows), is in relatively good condition considering the structural and moisture issues which have plagued the building through time. There are a number of locations where cracks are reflected in the plaster surface though generally of the hairline to the 1/16” wide variety rather than large gaps suggesting that the interior wythe of these thick walls remains relatively sound despite the dramatic issues at the core and outer wythe.

Fig.6

Timber Floor Structure and Flooring

As spec'd by Baird the floor joists were to be 3" x 14" and whitewashed in hot lime. Currently there is a range of joists sizes with a number of the heavier members approx. 3" x 12 ½" but many 2" x 12". There is a discernible bearing ledge at the top of each floor for the joist ends though they were always intended to bear further into the masonry than the narrow visible ledge.

Bearing directly on and into the stone makes the joist ends vulnerable to decay. There is obvious deterioration of the joist ends at several members at the second and third floors but, given the nature of the detail and likely extent of condensation at the stone walling, there also has to be an assumption of decay at other concealed locations.

While there are several locations where the 7/4" x 6 ¼" splined flooring is deteriorated it remains for the most part sound.

Fig.7: Note decay extending from bearing point at wall. Also lathing? nails

Windows

The original openings were pointed stone arches (lancets) in the gothic tradition. It is assumed that originally all contained lancet sash. There is one window opening at the ground floor (as well as the door) and three at the second while the remaining floors have four, one each at the center of each 'cardinal' elevation. While it is possible that openings have been fully infilled and hidden by plaster at the lower two floors it is also possible that it was always intended that the openings in these very thick walls at the base of the structure be limited. There are now a variety of conditions at the windows that appear to reflect various remedial structural measures and investigations.

The only window openings that have definitely been visible from the exterior since 1894 have been small rectangular openings in the shingled timber exterior at the west ('front') elevation of the upper floors. However, as viewed from the interior these openings still retain their Gothic arch, and surprisingly their upper sash, frame and casing. The lower section has been infilled with a metal panel and contains a vent. The window height from the floor was also modified and made shorter.

At the 3rd though 5th floors the other window openings retain their arched form but have either been partially infilled with masonry to form niches or completely infilled to the interior wall face. However a number of the latter have been partially reopened, perhaps in association with some of the past investigations, with the former infilling stones now sitting within the opening.

This changes at the second floor level where, while the pointed arch is still present at the outer wythe, the interior two thirds of the opening have been modified. This modification

Fig.8: Interior treatment at west elevation. Note surviving upper sash.

involved the setting in place of a heavy timber lintel supplemented within the depth of the opening with a lighter wood lintel to carry stone for the purpose of infilling the remainder (upper two thirds) of the opening. Whether any of these modified rectangular openings were intended to have window sash is unclear. More likely they became simply niches. It seems that the rationale for this reduction in opening size and lintelled treatment was to further improve the structural cohesion of the walling at the base of the tower. Thus it may have been undertaken as part of the 1894 work. That most of the window infill and modification work was undertaken from the interior is evident in that the exterior pointed arch remains in place at the original stone face of the building (confirmed during exterior investigation).

Unfortunately the timbers inserted as lintels directly into the masonry within such a high moisture environment have seriously decayed and been infested with carpenter ants.

As noted above it was found during exterior investigation that the arched masonry opening was still evident from the exterior suggesting that the infilling occurred from the interior.

Fig.9: Decayed timber lintel at lowered openings - 2nd floor

<u>Basement</u>

The basement is reached via a hatch which may be original. The stone coursing is regular and the stone units are squared – lime washed but not heavily plastered. There is no evidence of significant cracking on the interior face.

The floors structure is of heavy timber beams ranging from fully squared timber to a few members which remain three quarter round. While some members have been hewn others have been reciprocally sawn. As elsewhere they extend directly into the stonework. At some point they have been treated with creosote which accounts for the extreme odour when the hatch access hatch is opened. Despite the application of that strong preservative several of the members do exhibit decay though less than might have been expected.

It seems that the basement may actually be relatively tall but it is so full with debris from past site maintenance projects, oil and paint cans etc. that it is barely a crawlspace.


Fig.10: Note creosoted ¾ round timbers and quality of stone coursing

<u>Stone (See also Structural Analysis and associated Sketches 1 - 7)</u>

The central issue in all discussion of the future treatment of the building is the condition of the stone walling and attempting to ascertain the state of the walls has been the focus of this investigation.

As with most 19th century construction the wall section is comprised of an exterior and interior wythe of solid stone with a 'rubble' core between. This is the 'hearting' described by N.H. Baird in his specification. Ideally bond stones extend between the wythes, or, in the case of the thickest wall sections at least lapping each other deep into the core. This scenario works surprisingly well if the core is able to basically remain dry and stable but if subject to major incursions of moisture - becoming a wet mass unable to dry out and subject to freezing, as well as to settlement, the walling system breaks down. (In buildings such as this, of substantial wall thickness, evaporation is a very slow process made even slower by the already humid interior and exterior environments.) This involves a complex of negative impacts including: cracking of the bond stones; outward pressure on the outer wythe from frost and outward pressure on the outer wythe from settlement of the core. This, in turn, leads to cracks and displacement at the face stone and open mortar joints subject to the incursion of wind driven moisture. From first hand contemporary reports we know that the initial 'breakdown' of this system occurred almost as soon as the building was completed and more or less continued until 1895 when the wood girt stabilization and shingle weather protection were introduced.

The problems of the building were further exacerbated by the original, relatively tall, Gothic openings stacked vertically at the cardinal points of the elevations. In the face of the extremely heavy wind forces often in association with driving precipitation and particularly as the core of the wall deteriorated, the area from the point of one arch to the sill of the opening above became a line of weakness eventually virtually separating the elevation into two halves. The openings themselves contributed to the problem as moisture would collect on the sill and, with the masonry not well maintained, would migrate through open mortar joints into the wall section.

Thus cohesion was being lost both through the wall section and across the wall. However, interestingly enough, relatively little of this lack of cohesion was translated through the whole wall section on to the interior face of the stone.

The test 'windows' opened up at the exterior and the associated borescope probes undertaken at both the exterior and the interior essentially illustrated the above. The probes clearly showed the lack of cohesiveness of the core, largely void with, occasionally, just shards of stone and/or clumps of mortar visible. Typically the edges of the stone units seemed 'clean' as if they never had been in contact with mortar fill (or perhaps the original edges had spalled off at some time.) It should be here noted that the two probes undertaken from the interior of the first floor could not actually reach the core due to the wall depth (approx. 6'). Where visible the extent of the void and lack of evidence of mortar on the adjacent stone was particularly noteworthy, as if it had never been filled, in direct contrast to Baird's spec. If that is the case it would be another example of the poor workmanship/supervision of the original construction as descried by Dunlop when he first inspected the building.

While the area of face stonework made visible within the sample 'windows' was necessarily small important information was gleaned. It was confirmed that the stone walls had been limewashed with the coating having a discernible thickness, approx. 1/16". This is not surprising given typical 19th century practice as the limewash acted as a further form of weather protection and the white colour made the whole lighthouse stand out as a landmark when viewed from a distance. With the lime wash mortar joints and coursing were difficult to ascertain. However two of the 'windows' were located in reasonable proximity to corners and in those areas it seemed that ashlar blocks did make


Fig.11: Borescope probing at revealed mortar joint. Quoin masonry appears regular.

up a properly quoined corner and these were relatively intact**.** By contrast, at the probe set at the approx. middle of the elevation (both in terms of height and length), a large crack of varying width, essentially vertical (though meandering) and exhibiting displacement was evident. Examination of the immediate area (below the actual opening) confirmed that the crack was extending from, or in close proximity to, a lancet arch (Fig.11). Of interest, as alluded to earlier, the masonry opening (m.o.) of the window was still discernible as a niche, i.e. had not been filled in to the wall face.

In summary the results of the investigation, though limited, appear to confirm a scenario in which a dangerous level of discontinuity existed, approximately in line with the windows, over much of the wall height across each, or at least several, elevations particularly as orientated to the prevailing wind direction. This major failure, coupled with other cracks, bulges etc. was obviously of a level of significance, that the late 19th century engineers/experts considered it unstable and requiring the extensive stabilization treatment which has been in place since that time. **See Structural Analysis section.**


Fig.12: Serious vertical crack and displacement extending from lancet top

<u>Timber Girts and Wood Shingle Cladding</u>

In 1894-95 the lighthouse walls were stabilized with a system, which, according, to original reports at the time, included iron bands and a system of timber girts fastened to wood firring. This assembly was then clad in wood shingles to protect the stone and timber from the weather.

While none of the modern investigations (1982 – present) have revealed the iron bands, the timberwork remains intact and the system has been extremely successful at both stabilizing the structure and protecting the stonework from further deterioration.

In the areas opened up for the recent investigation the timber was sound with no evidence of deterioration/decay. Horizontal boards were secured to 2" x 4" sawn wood furring. Surprisingly not all furring strips were continuous through the areas opened up for examination. Horizontal members were cross-lapped at the corners. The dimension and exact configuration of members varied depending on location. At the south/southwest corner and mid-height this consisted of full 2" x 4" sawn lumber ganged as four members with a 2" gap above and below". As part of this treatment a 4" section was carried across the corner as one laminated 4" x 4" (Fig.12**)**. At the middle of the south elevation there was a combination of 2" x 4"'s (3) and 2" x 2"'s (1) with no discernible gap within the area opened up. Toward the top south to southeast corner "2 x 4's alternated with 2 x 2"s in a solid assembly. Above (5th storey) it appeared to be solid 2"x2" members. It is likely that considerations such as the irregularity of the stone surface in different areas, the level of stability required in specific areas and the need for ventilation between the stone, the timber and the shingles influenced the range of configurations witnessed. Surprisingly no fasteners from the furring into the stone were encountered and all the fasteners uncovered were round wire nails, including the larger nails laminating the dimension lumber together. Nails such as these were more typical after 1905. No large bolts or lags with washers and nuts were found. Fasteners were iron and corroding.

Wood shingles comprise a number of generations but are sawn, appear to be approx. 18" in length with 5 " exposed. They have been heavily painted with the paint typically flaking, alligatoring and separating from the wood surface.

Fig.13: Various girt configurations are used with cross-lapped corbers

Metal Cladding at the Base

The base of the tower flaring out at the bottom over the top of the 'water table' is currently clad in painted galvanized sheet steel. The material is installed in the form of horizontal pans lapped in the direction of flow and covered with a 'hip' cap at each corner. Pans are fastened along their top and bottom edges by a line of exposed screws. It is not clear to what extent the fasteners actually penetrate the masonry. It is more likely that the panels are fastened to wood furring strips. The material is in relatively good condition with loss of paint at several locations but no significant corrosion. The upper pan has a cap which extends up behind the lowest shingle.

While it is unclear exactly when this treatment was installed it likely dates c. 1982 as many of the reports through the 1960's and 1970's commented on the ongoing replacement of shingles required at the base of the walls. This was no doubt due to the extent of wave related spray being absorbed by those bottom shingles. C. 1900 photos show that it was shingled to the stone base as part of the 1894 shingle cladding.

Stone Base and Foundation

The moulded stone base, plinth and foundation is the only area of exposed original stonework remaining at the building. Despite its somewhat deteriorated condition the cut stone base, plinth, foundation cap and foundation wall quoins do provide a glimpse of the impressive visual nature of the original work. Also it has to be acknowledged that, given the problems of the masonry in general, it is impressive that this area, subject to so much moisture including the incursions of surface water, is still as intact as it is.

However, the problems are still substantial. The top of each corner has been built up in concrete intended to improve drainage. It is not clear whether they have been cast against an original shoulder detail or are concrete right through. This may vary from corner to corner. The northwest corner is actually completely covered in concrete and/or cementitious parging which itself is beginning to crack. All these corners are beginning to pull away from the lighthouse walls with the northwest and southeast the most pronounced – with cracks extending through the projecting foundation cap course as well as the gap widening at the joint between the built-up shoulder and the octagonal wall common to all the corners. There are many vertical cracks through the plinth block, particularly at the front. The rubble stone of the foundation walls is also unstable at the southeast corner. The cardinal foundation elevations that are in line with the walls above fare better though there is substantial cracking at the front step of the entrance and at the plinth along the front (west).

Fig.14: North-west corner

Structural Analysis (Sketches 1-10)

In developing recommendations for KOPPLA regarding the future approach and stabilization/conservation/restoration treatment of the Lighthouse it was considered essential to undertake a comprehensive structural analysis of the site both in its current timber stabilized form and as a stone structure without the girting and cladding. The exploratory 'windows' and borescope probes were utilized to provide the background information necessary for such consideration.

Structural analysis of an existing building generally involves an examination of the building materials, the layout of the structure, the environmental loads in the area in which the structure is constructed, and the possible use loads proposed for the structure. For the Presqu'ile Point Lighthouse, the construction of the original building was stone masonry, a common building material in this area of Ontario, and there were a number of experienced, well-qualified contractors who could bid on this type of construction. Normal stone masonry construction for this period, was done in several wythes (thicknesses) with the exterior or outer wythe being constructed in well fitted stone units (usually cut to near rectangular face) and with various depths. The pattern or "bond" was usually the choice of the mason, and often reflected where, or under whom he was trained. The size, quality and quantity of stone units available in the immediate area also were significant in determining the style or appearance. In the case of this lighthouse, the stone appears to be local limestone, either from the adjacent beach, or quarried from nearby. Typically the outer wythe was comprised of larger stone units, while the other wythes, usually not seen by the public, used the smaller stone. The interior wythe was often as well laid as the outer wythe, but often with less concern regarding aesthetic appearance. Filling the section between the outer and inner wythes was a 'rubble' core consisting of material often not formally laid up at all. It included the cut away pieces of stone from the trimming of the stones in the outer and inner wythes, scrapings of old mortar off the masons mortar board, small stone that wouldn't fit the outer or inner wythe, and stone judged to be substandard by the mason on site. The core was often not very strong, but usually was self supporting. The purpose of this core was to transfer stresses induced in either the inner or outer wythe to the opposite side and thus even out the stresses from applied loads, so that at the foundation level, the masonry wall imposed an even load on the bearing soil or bedrock. For most residential housing units of one or two stories, this method of construction was adequate and most have lasted well over a century with little or no major structural work required. For residences, loads on the walls were from the self weight of the wall itself, the weight of timber framed floors and roof, and the loads imposed by the residents and environment (furnishings, people loads, roof snow loads, wind loads, and rarely, seismic loads). For the lighthouse, there is such a small roof area, that snow loads are insignificant. The people loads were limited to service personnel to maintain the light, and this too was insignificant. The interior structure was a timber framed floor and access ladder to each of the floors. Compared to the self -weight of the masonry walls, this load from the interior structural elements was also insignificant. The only really significant loads were environmental, primarily seismic and wind. Their analysis follows:

Seismic Analysis:

The area of Ontario where the light house is located is rated as relatively low for seismic activity as are most of the communities along the north shore of Lake Ontario, and up the Trent River system. However, because of a potential danger to the public should the lighthouse collapse, it should still be analysed. In its present configuration, the Presqu'ile Point lighthouse cannot be analysed with any certainty due to its unusual construction. Essentially, the lighthouse structure is a stone masonry tower with little or no inherent strength in the masonry, all held in place by horizontal tension bands of laminated timber extending from just above the stone masonry base to the top. There is no evidence of a positive connection of the timber framing to the masonry. The actual timber framing is much like the timber corn silos commonly constructed in Ontario at the turn of the last century, although these silo structures were often built with integral vertical members which tied the horizontal tension members together. The vertical members in the case of the lighthouse do not appear to be well connected to either the masonry or to the horizontal timber members. The horizontal timber members form an almost continuous confining cylinder with the spaces between members varying, but in no areas less than 25%. Openings for windows on the west facade have been properly famed to distribute stress to adjacent members.

The lighthouse configuration was first analysed as a timber silo supporting a coarse granular material with a very high angle of repose and high gross weight. It was determined to be stable in these conditions. It is obvious that this analysis is flawed as the stone masonry is self-supporting and its behaviour during a seismic event will probably not be that of loose granular material.

In examining and analyzing the structure as an unreinforced stone masonry tower, there are a number of problems that would indicate that the tower is unstable, particularly during a seismic event. As a masonry structure, there are two concentric cylinders with the inner cylinder being circular of constant inside diameter (3.6m (12 ft.)) and an outer octagonal cylinder with outside face to face dimensions varying from 6.5m (21.5 ft) at the base to about 5.15m (17 ft.) at the top. The inner cylinder is well constructed stone masonry, some 200mm (8 inches) thick, with the larger stone masonry units approximately 300mm wide x 200mm deep x100 mm high (12 inches wide by 8 inches deep by 4 inches high). The lime mortar has a low strength of less than 1000 psi which is not unusual for masonry of this period. The outer octagonal cylinder is constructed in similar material, although the masonry units near the base appear to be larger (300 mm wide x 250mm deep x 150 to 200mm high (12 inches wide x 10 inches deep x 6 to 8 inches high)). Quoins at the corners at least at the lower levels appear to be almost as originally specified (355mm thick x 700mm wide (15 inches in thickness x 28 inches wide)). The space between the two cylinders (the core or "hearting" as referred to in the original specifications) is just broken lime stone of varying sizes, but most appear to be less than 200mm (8 inches) on a side. (see Sketches #1, 2, & 3) These same specifications called for the hearting stones to be laid in well made mortar. There did not appear to be any evidence of mortar in several of the inspection bore holes and only some

staining of what might have been mortar on a few stones in the other bore holes. The inner wythe or core is unconsolidated rubble and does not contribute to the strength of the tower in any way, so that the outer and inner wythes (cylinders) while supporting the vertical weight of the tower, also provide the confining strength keeping the loose core stones in place. (See Sketches #4 & 5 for original specified construction techniques)
In a mild seismic event, vibrations (lateral and vertical movement) would cause these loose stones to exert pressure against the outer and inner wythes . The outer wythe would thus be acting in tension, and the inner wythe in compression. Stone masonry acts very well in compression, but poorly in tension and it is likely that vertical cracks would open in the outer wythe in the weakest areas (probably between the vertically-in-line windows – (See Sketch # 6)). The existing timber cladding acts as a confining shell, and while cracks may occur in the outer wythe, there has been little or no displacement of the stone masonry. It is unclear whether the addition of timber cladding had this intention in the 1894 rehabilitation of the lighthouse, but it certainly has performed well as a structural system, keeping the tower intact.

In a strong seismic event, the tower would experience lateral swaying, which would set up vertical shear stresses. These shear stresses would concentrate at the tops of the gothic arches of the windows and door, and a crack would probably develop and move vertically through the windows. (See Sketch # 7) These cracks would show up on both the exterior and interior wythes. The infill of these windows on the north, east, and south walls would probably not alleviate these stress induced cracks as the infill masonry is not structurally keyed into the original masonry. Similarly, the timber cladding on the exterior, while providing a confining shell for the tower, is flexible enough so that it would not prevent side sway, and cracks in the masonry would still be expected in the event of a major shake. Cracks through the tower as a whole, induced by high seismic forces are a major weakening of the structural system, and there could be partial collapse. Extensive repairs would probably be required to stabilize the structure. It would appear that there have been no major seismic events in the last 176 years as there are no visible seismic type cracks on the interior of this building.

Prevention of Seismic Damage:

If it is assumed that there will be no major seismic events in the next half century (assumed life span of the timber cladding), then the lighthouse is structurally stable in its present form.

If there is an intention to change the present structure (i.e., removal of the timber cladding; - reopening of all or many of the windows; - adding a heavy load to the top (cast iron light housing)) or use (ie allowing general admission to the public; - allowing public gatherings around the immediate vicinity) then the tower will require structural upgrading to make it stable and safe.

Grouting:

The unconsolidated rubble core appears to be the cause of most of the structural shortcomings. To stabilize the core, a grout needs to be inserted that not only fills the voids between the loose stones, but also has enough inherent strength and adhesion to cause the outer wythe, the inner wythe and the core to act as one unit. There are a number of grouts available, but the characteristics required for this structure will limit the choices to only a few. These characteristics are as follows:

1. Low viscosity - so that it flows through relatively small spaces and fills all voids ; - the grout should be easily pumped or poured, and can flow into cavities by gravity alone; -
2. Good adhesion – so that the connection of mortar to the masonry units is strong enough for distribution of stresses from one masonry unit to the next adjacent unit; -
3. Relative high early strength – so that the grout stiffens up enough that there are no long delays in between pours; - ordinary lime can take up to two or three weeks to get enough strength to support loads without deflection; - hydraulic limes can exhibit the same strength in a few days ;-
4. Long term "softness" or ability to resist stresses without fracturing – so that the tower can withstand sway and vibration without cracking. Most Portland cement or epoxy cement based grouts while being very strong, are also very brittle and fractures are permanent;-
5. Exhibit "autogeneous healing" – that is, if and when the cementitious material in the grout leaches to the interior or exterior surface with water vapour transmission, then it will recrystalize in micro cracks and on the surface when the moisture evaporates, and some strength is retained ; -
6. Allow for water vapour transmissibility – as wind driven precipitation enters the masonry, it can be absorbed by the whole depth of the masonry; - as drying conditions occur the moisture can move back out with no trapped moisture causing surface spalling or frost damage ; -
7. Low Cost – given the extent of voids in this tower, there will be a large quantity of grout required; - The grout must have a relatively low cost for the materials as well as being simple enough to be prepared on site, and installed by local craftsmen.

A quick search for ready mixed grouts on the market revealed only one grout that fulfilled most of the above requirements, and that was “Flowmix” a NHL type lime based grout. Unfortunately, it does not meet the seventh requirement in terms of material cost, but can be installed by competent experienced masons.

A good quality grout can be prepared on-site, using screened sands, NHL limes, plasticizers, and a high quality mixer. While this reduces the material costs, the labour costs will increase because of the time for preparation.

Installation of the grout requires close supervision, and the ability to stop operations on a moments notice. If possible, grouts should be installed and allowed to fill voids by gravity flow as opposed to pressure grouting. Even then, very high lateral pressures can occur as this “heavier-than-water” liquid is poured into a wall. For example, one metre (3.3 ft.) depth of grout exerts about 19.2 kN/sq.m (400 lbs/sq.ft.) laterally against the outer or inner wythe of masonry before it hardens up, and this may be enough to bulge or burst the wythe. Before commencing grout activities, test should be conducted to determine the time required for the grout to set enough to sustain vertical loads from another application of grout. Similarly, tests may be conducted to determine the safe height of each pour to avoid bulges or collapses. As grout can escape from relatively small cracks or holes, then enough exterior cladding must be removed so that access to cracks is possible and cracks can be plugged or sealed before the pouring of grout. In applying the above recommendations to the lighthouse, it may result in the removal of the exterior timber framing in stages as the grouting progresses from the bottom to the top. This will need to be determined on site at the time of the grouting operation.

Steel (Stainless) Reinforcing:

Even though unreinforced towers have stood for centuries in earthquake prone areas around the world without collapse or damage, the Canadian building codes require a minimum amount of reinforcing to withstand seismic events for new masonry towers. Trying to retrofit an old masonry tower to meet these requirements would be almost impossible to meet without total dismantling and reconstruction. In our analysis of this tower assuming that the loose stone in the core has been consolidated with a medium strength grout, there was good seismic resistance to overturning, but the vertical line between windows is a weaker plane for shear and is still a concern for cracking during a seismic event. It is possible to place reinforcing steel in areas most susceptible to seismic damage, and to mitigate some of the potential damage. A general rule for placing of reinforcing steel is to intercept potential cracks or fractures at right angles. Based on the drawing showing the vertical cracks through the windows, then steel should be horizontal, embedded in the masonry between the windows. The most popular way to insert steel in stone masonry is to drill a hole into the masonry and then to insert steel bars in the hole and grout around the bars. One of the most efficient ways of inserting steel bars is to drill the hole, and then insert a bar surrounded by a sock of expandable geotextile material. Once the bar is in place, the sock is pumped full of a very strong grout which adheres to the bar as well as the sides of the hole. These reinforcing units are manufactured by a company called “Cintec” and specialist craftsmen are required to do the installations. These units work, but are highly dependant on the skills of the craftsmen. They are also very expensive. One requirement of using the "Cintec" anchors is that the masonry be consolidated, so the grouting of voids is a necessity. The holes must be drilled using a diamond core bit and after the drill rod and core is removed, the hole must stay open for a period long enough to insert the sock enclosed reinforcing rod. Boring the hole requires stable scaffolding (to mount the drill rig) and the diamond bit requires a pressure water source for cooling. Obviously, this cooling water saturates the wall in the vicinity of the hole. Therefore the process must occur well in advance of freezing weather, so that the wall can dry out before it suffers from frost. Sketch # 8 illustrates a horizontal section on a plane between the windows and the possible placement of a set of Cintec anchors. Sketch #9 is a vertical elevation showing the vertical placement of the reinforcing bars in the north wall. The decision to reinforce is an economic one, as without reinforcing, the tower will likely remain intake after a severe seismic event, but will require extensive repairs. If it is reinforced, (at considerable expense) , then it should suffer little or no damage through a similar seismic event.

Other Structural Reinforcements:

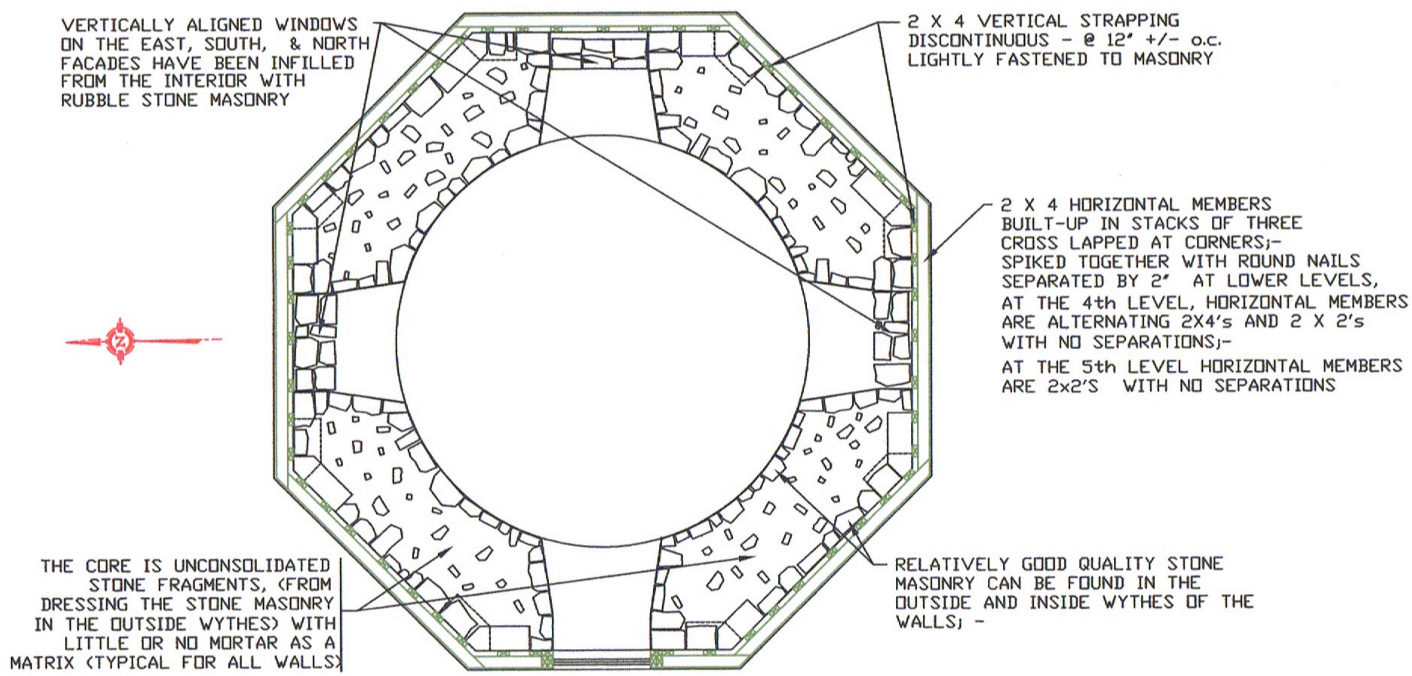
As it appears that the vertically-in-line windows produce a weak plane in the vertical walls, then if the window openings were reinforced to transmit stresses, then the openings would have little negative effect on the structural capacities of these walls.
In the present configuration, the masonry filled-in openings on the upper levels are not keyed into the surrounding masonry, and hence do not contribute to distributing vertical and horizontal forces into the main structural system. For the lower two levels, timber lintels were inserted into the opening, thus providing some distribution of stresses, but these lintels are prone to rot and at this stage, are largely ineffective. The surest way of reinforcing the infilled openings would have been to cut out vertically alternate quoins around the opening, and then to key in the infill masonry. A simpler, but not as effective way would have been to grout in stainless steel or bronze rods into the masonry around the opening, and then to infill with stone masonry, encapsulating the rods.
Alternatively, without closing in the openings, a ridged, rust resistant steel frame could be inserted into the window or door opening, and then deeply (150 – 200mm) fastened into the surrounding masonry . (See Sketch # 10)

In other towers where reinforcements have been successful, the whole tower has been reinforced on the interior with a structural steel frame. These frames have varied from a lattice work of relatively small members that were taken piece by piece inside, and erected on the interior to one that involved removing the roof, and lowering in a steel centre column with spiral staircase and steel platforms. There is a danger of over-reinforcing, where the steel reinforcement is such that when the steel moves differentially from the surrounding masonry through extreme temperature changes, the steel frame literally tears itself away from the masonry, or breaks up the masonry. These systems must be carefully designed to avoid these problems.

Wind Load Analysis:

Presqu'ile Point is one of the windiest places in Ontario, and the lighthouse bears the brunt of these forces. The prevailing wind in the summer months is from the south west, and in the winter, it is from the north west. Spray from large breaking waves on the shoals to the south west can saturate the windward surface of the tower even on a clear day. During the winter months, with the prevailing NW winds, there is a much shorter fetch, the waves are not as high, so that the spray and hence icing is not as severe as might have been supposed based on this exposed location.

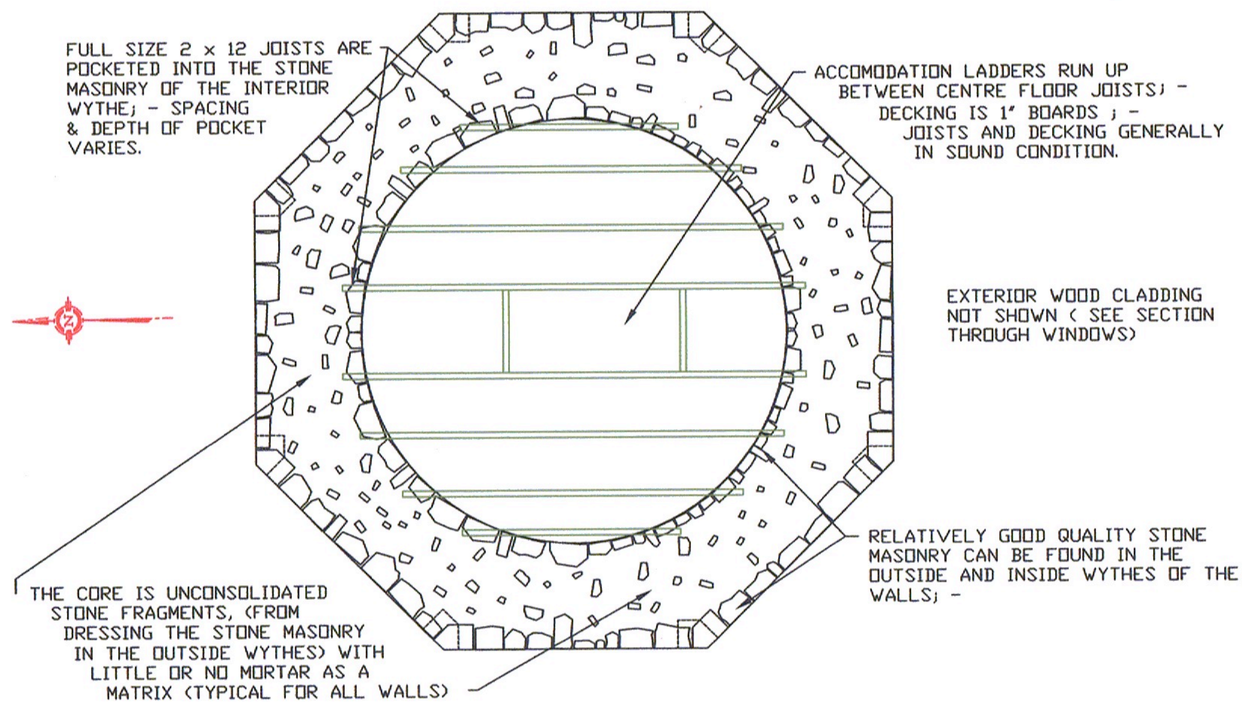
Tall cylindrical structures are subject to direct pressure on the windward side, and a negative pressure on the leeward side. The sum of these two pressures over the exposed area of the tower, creates an overturning moment at the base. Analysis shows that the overall weight of the tower more than counteracts this overturning moment, and there is no danger of toppling. However, in addition to the winds producing pressure on the flat surfaces on the windward and leeward sides of the structure, the effect of the wind moving around the tower produces what is referred to as "vortex shedding" which produces pressures acting at right angles to the direction of the winds. These pressures alternate from side to side and in severe wind conditions will cause vibrations. As previously discussed, vibrations cause the loose core of broken stones to exert interior pressures against the outer and inner wythes of masonry, producing cracks and bulges in the outer wythe. In its present condition, with the timber frame shell, these forces are very much dampened, and the most damage appears as plucked shingles from the sides not facing into the prevailing winds. Calculations show that if the timber cladding were to be removed, and the loose stones in the core were consolidated with grout, then the lateral forces causing vibrations would have little or no effect on the structure.



SECTION THROUGH WINDOWS 3rd FLOOR

AS FOUND 2014

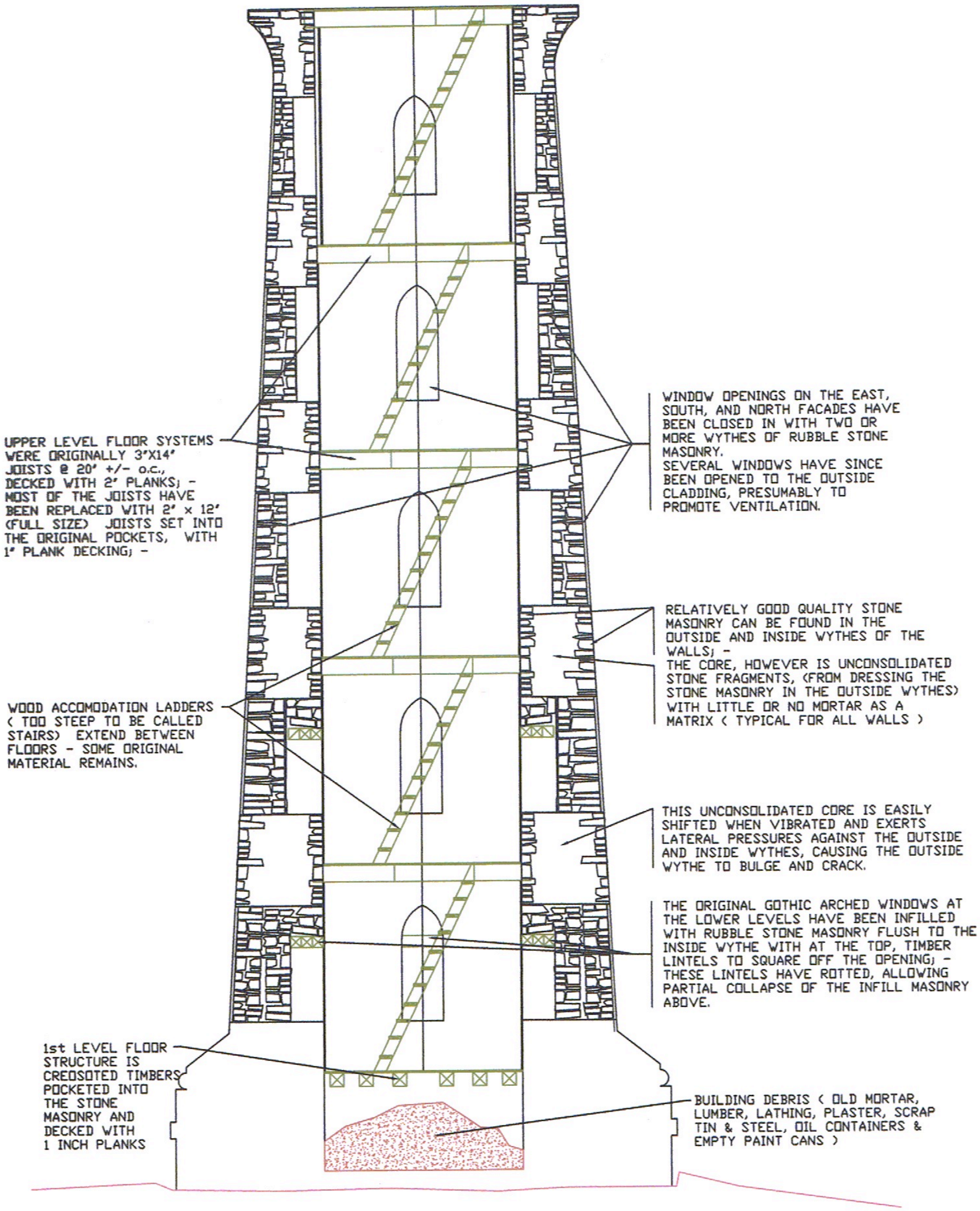
SKETCH # 1



SECTION THROUGH FLOOR STRUCTURE 3rd FLOOR

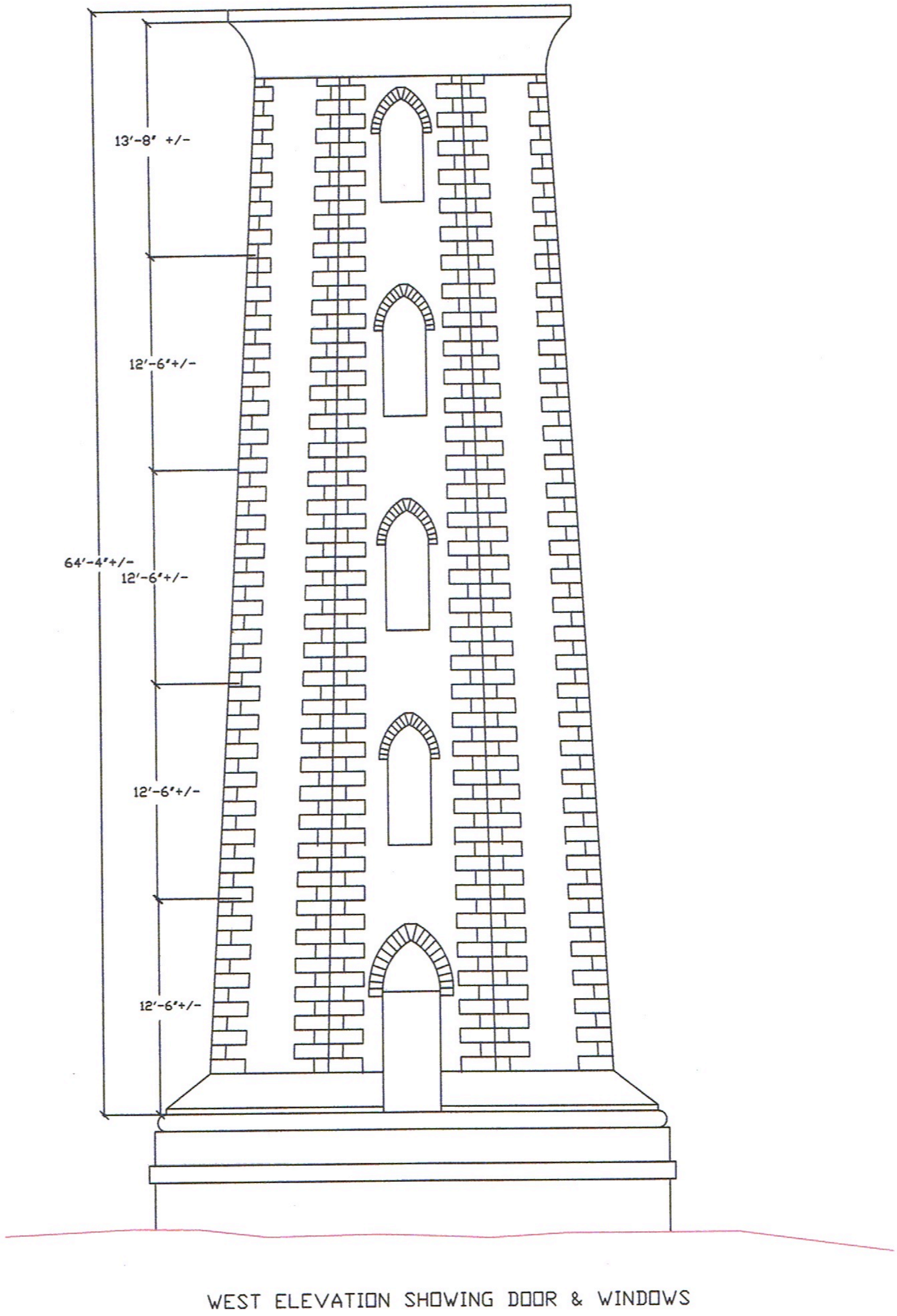
AS FOUND 2014 (outside cladding not shown)

SKETCH # 2



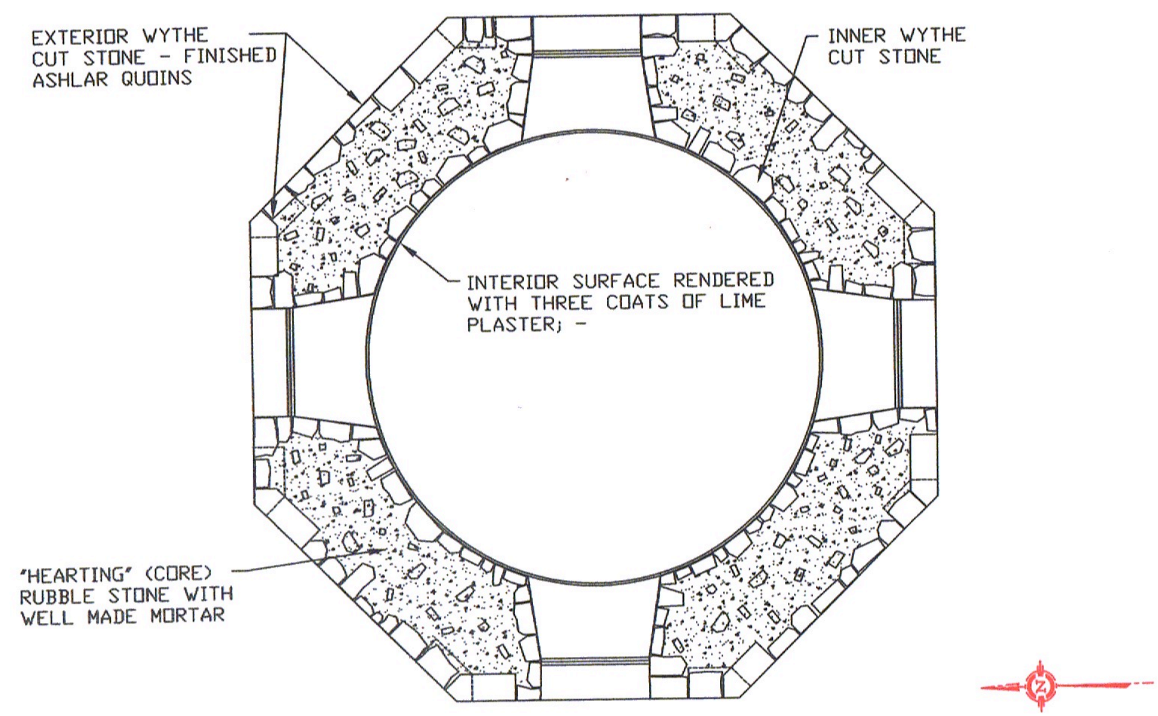
VERTICAL SECTION THROUGH THE NORTH - SOUTH WALLS
ILLUSTRATING THE CLOSED-IN WINDOWS AND THE
AS-FOUND MASONRY CONDITIONS (OUTSIDE CLADDING NOT SHOWN)

SKETCH # 3



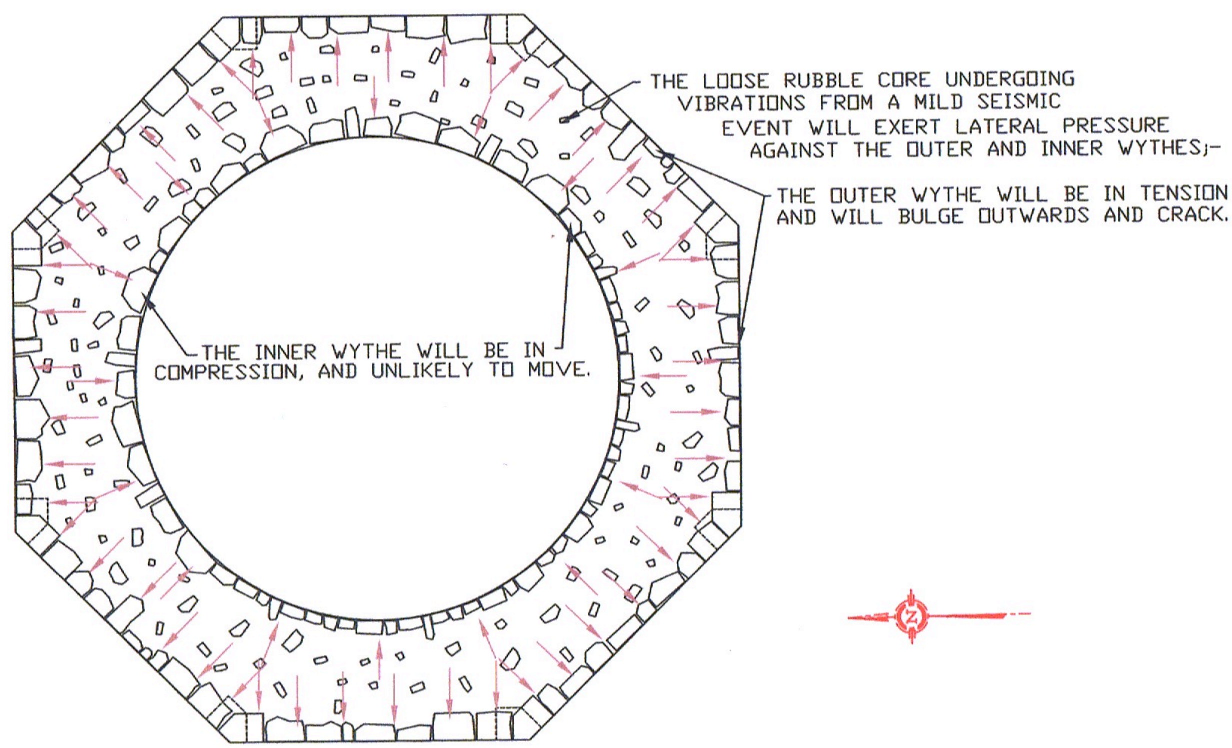
WEST ELEVATION SHOWING DOOR & WINDOWS
AS SPECIFIED 1837

SKETCH # 4



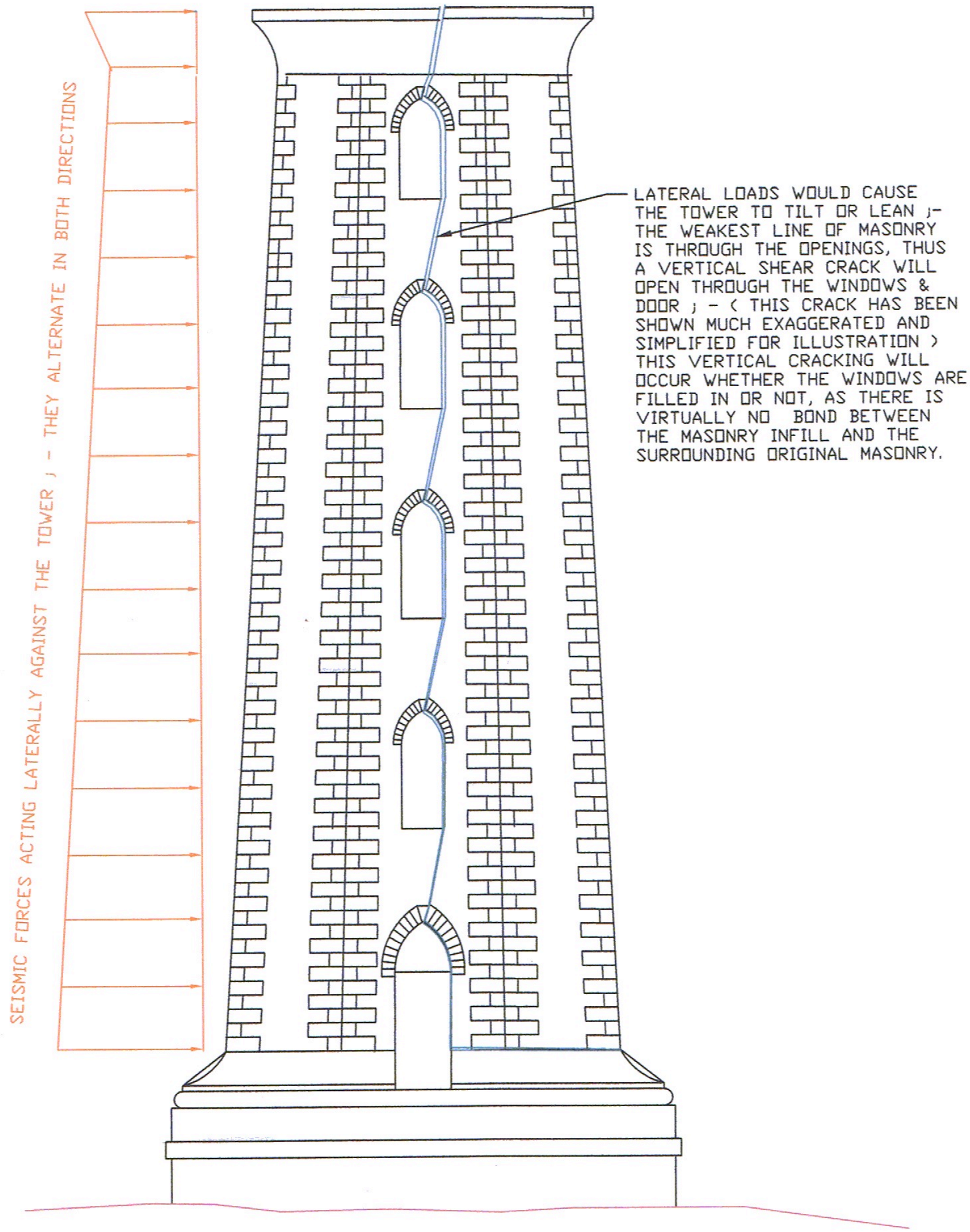
MORE OR LESS AS SPECIFIED 1837

SKETCH # 5



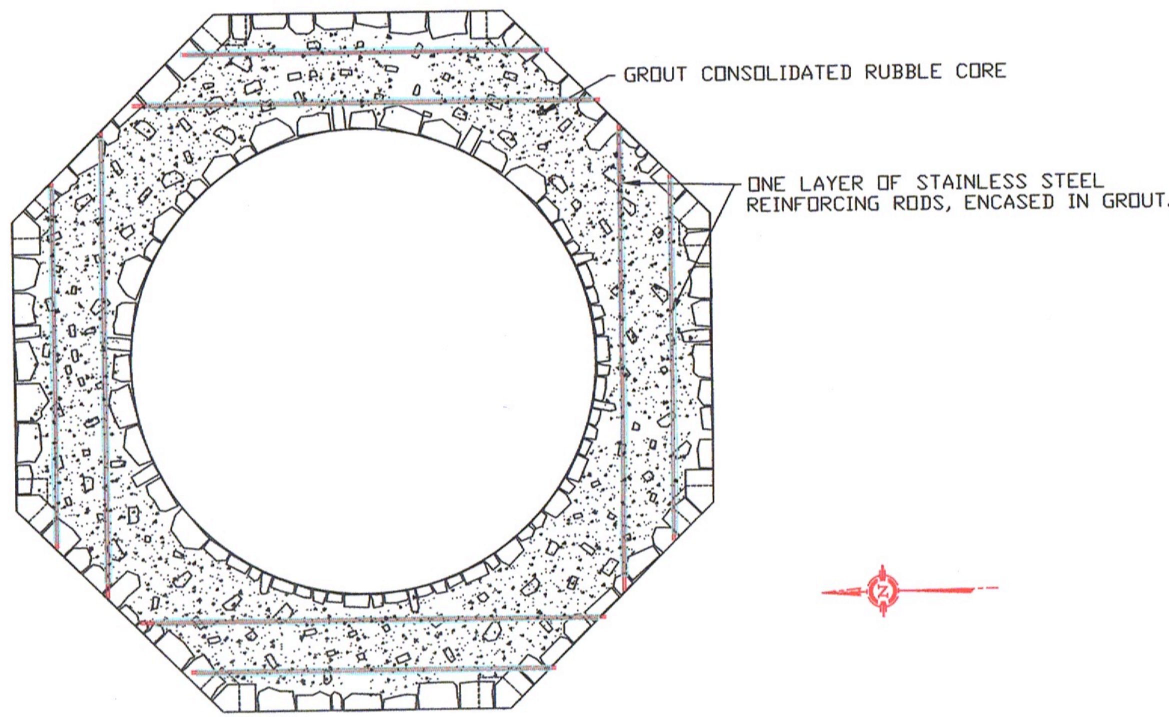
SECTION THROUGH STRUCTURE SHOWING LATERAL LOADS FROM THE UNCONSOLIDATED CORE

SKETCH # 6

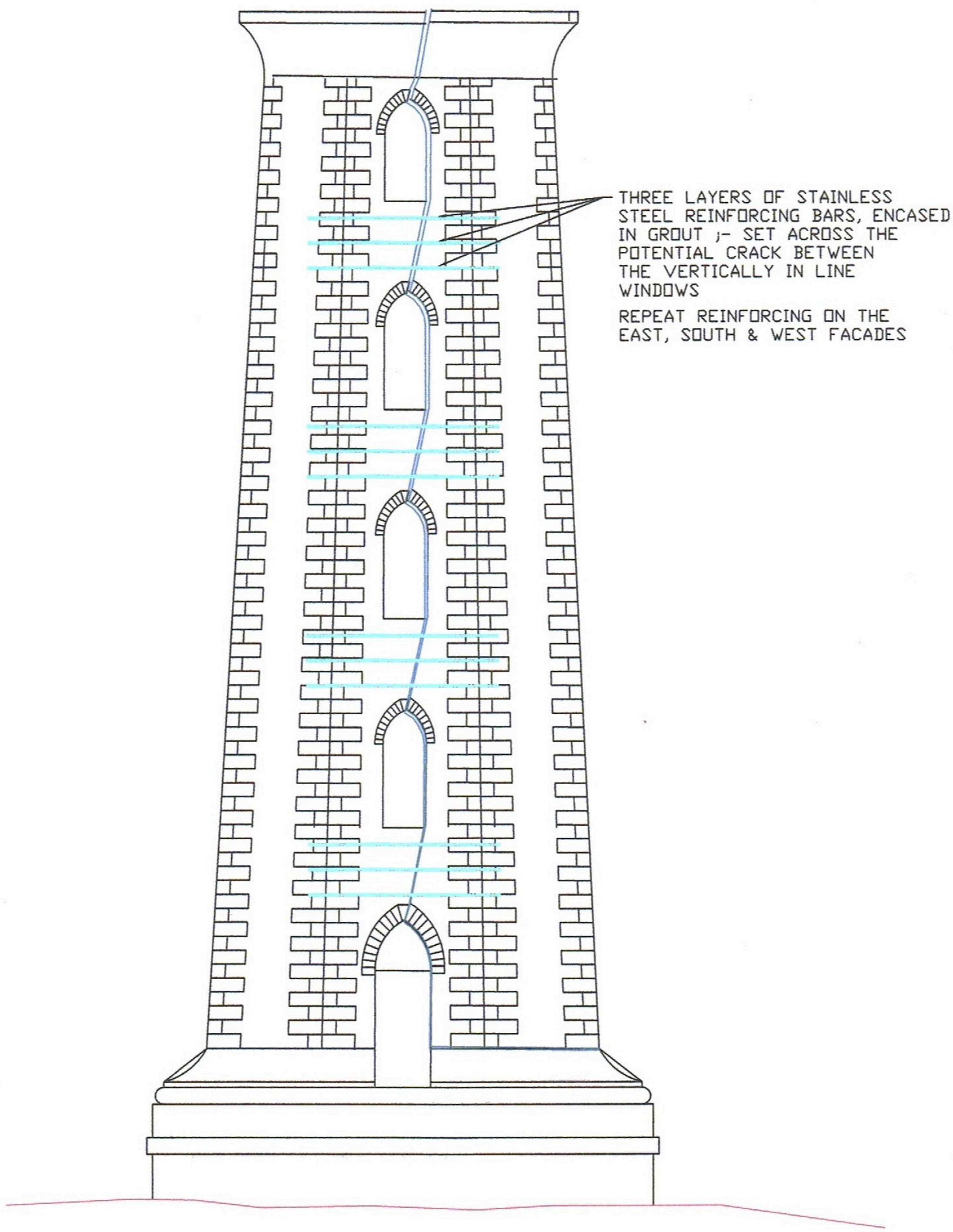


WEST ELEVATION SHOWING DOOR & WINDOWS ILLUSTRATING THE EFFECTS OF A SEVERE SEISMIC LOAD ON THE TOWER.

SKETCH # 7

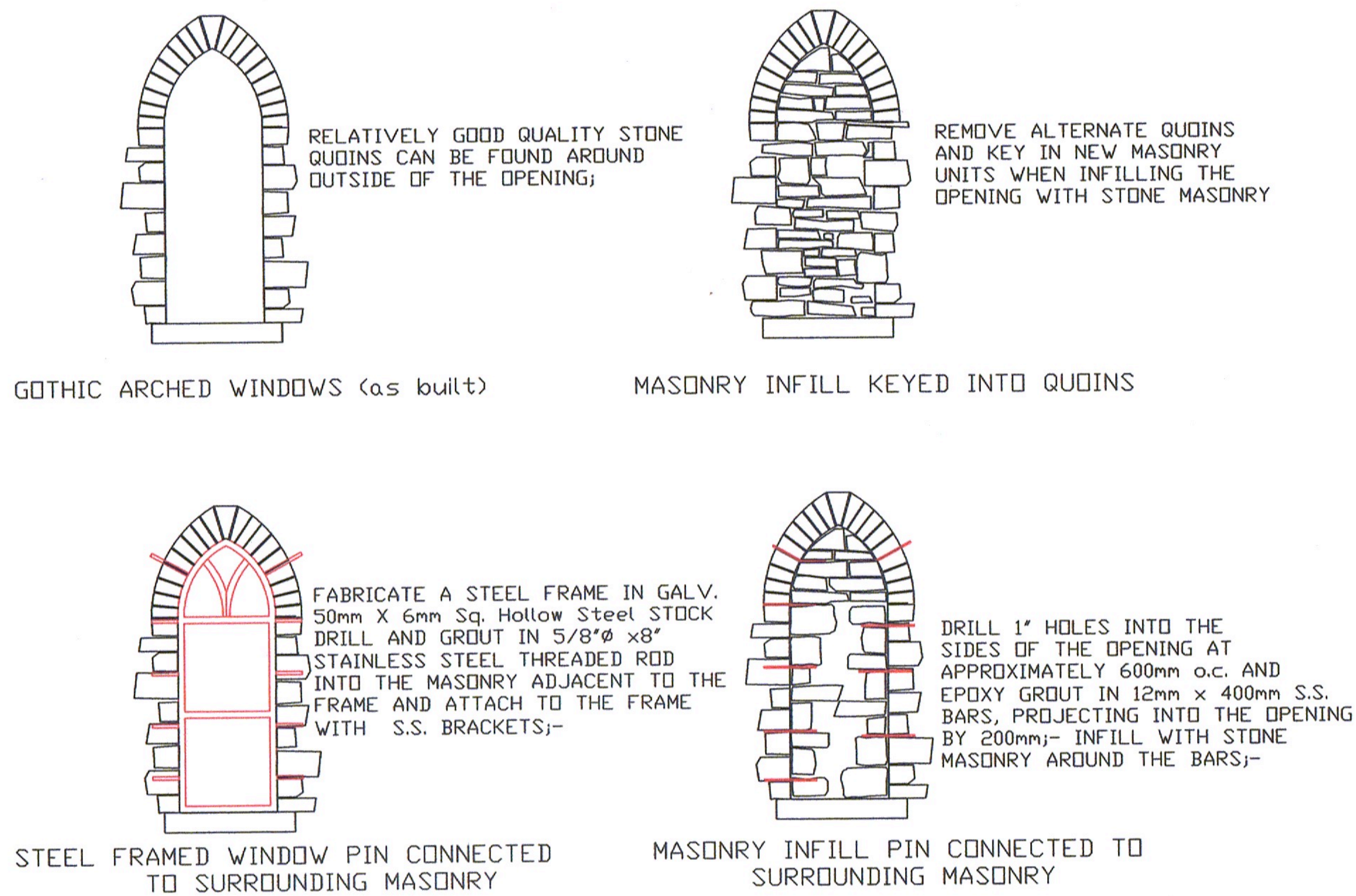


SECTION THROUGH STRUCTURE SHOWING HORIZONTAL
CINTEC TYPE REINFORCEMENT BETWEEN WINDOWS
SKETCH # 8 NOT FOR DESIGN.



WEST ELEVATION SHOWING PROPOSED HORIZONTAL
CINTEC TYPE REINFORCEMENT ; -

SKETCH # 9 NOT FOR DESIGN



ALTERNATE METHODS OF INFILLING WINDOWS TO RETAIN OVERALL WALL STRENGTH

SKETCH # 10 NOT FOR DESIGN

Summary of Structural Analysis:

1. In its present configuration, the lighthouse is stable under all conditions except for a severe seismic event. This area of the country is not prone to seismic action and there is no evidence of seismic damage to the tower,(based on an extensive survey of the interior, and a limited survey of the exterior). Prediction of seismic activity is not an exact science.

2. The timber cladding is essential for the structural stability of the tower as the stone masonry was never stable even when first constructed. If the masonry portion of the tower had been built as originally specified, it probably would not have needed the structural intervention of the timber cladding so early in its history. The main problem is with the unconsolidated core, which shifts with natural vibrations and applies outward pressures to the outer and inner wythes of stone masonry.

3. Should the decision be made to change the structure i.e., removing the cladding; - adding a weight to the top such as a cast iron cupola and light; - opening up most of the windows; - then there will be a requirement for structural intervention. Similarly, if there is major change in use such as allowing access to large numbers of visitors or constructing structures for public use around the base, then structural interventions will be needed for public safety.

4. As the prime reason for the bulges and cracks in the tower is the unconsolidated rubble in the core, then, if the cladding were to be removed, a grouting procedure is required to unify the inner and outer wythes with the core. The grout should be selected with care, as this intervention will for all intents and purposes be totally irreversible. With a consolidated core, the tower will be stable without the cladding, except in a severe seismic event.

5. In a strong earthquake, even with a consolidated core, there will be heavy damage to the structure, with possible partial collapse. There are several options to reinforce the tower for such an event, although costs for this work will be significant. Any one or all of the following may be employed:

i.) insertion of steel reinforcing through the known weak points of the structure

ii.) reconstruction around the window openings to reduce stress concentrations and to make the windows carry some of the loads.

iii.) construction of a new interior structural frame to provide lateral support to the masonry as well as improved access to the upper levels

While the above interventions are costly, they may be less expensive than repairs to the tower, should they not be employed.

6. While the changes mentioned above will have a positive effect on the structural response to earthquakes, they will also have an effect on maintenance costs. Removing the cladding will necessitate relatively frequent repointing and rendering of the stone masonry. Reinstalling windows will create areas for weathering and ingress of water unless inspections and maintenance of these openings is vigilant. Solid masonry takes considerably longer to dry, so that it is essential that leaks be eliminated as much as possible to prevent frost damage to the consolidated stone masonry. Insertion of steel (other than stainless steel) as an internal structure requires frequent inspections and maintenance to prevent rusting or corrosion.

Options/Recommendations

While there are many technical conservation items that will be common to any approach to restoring the Lighthouse the first crucial decision is in regard to the overarching vision for the future of the historic site. There is a 'world of difference' between an option which, essentially, conserves the 1894-95 timber stabilized lighthouse, and that which attempts to restore the building to its original form/appearance with stone walling exposed. These two major options are discussed in more detail below. It should be noted that the reinstatement of a lantern would be an acceptable element in either scenario having been present through both iterations of the building until removed in 1966.

Fig.15: PPL c.1900

The present timber girt shingle clad version of the structure has stabilized and weather protected the unstable stone structure since 1894 – for 120 years. Conversely the original stone structure appears never to have been completely stable (due to the level of workmanship/quality control/detailing) and began its deterioration almost from the moment it was completed. It remained in its original form only for 54 years before having to be wrapped in timber. Thus, while acknowledging the interest inherent in the original appearance of the building the shingle clad treatment must definitely be considered to be historically authentic to the historic site and with perhaps an even greater claim for being preserved.

Where examined the timber members were found to be in good condition and the painted wood shingles, while of several generations, were still acting as effective weather protection beneath a flaking, weathered paint coat. In broad terms the major cost in undertaking the preservation of the building in this form would be renewing the shingle cladding (though a phased approach would also be possible.) Again by contrast the full removal of the timber protection would require not only extensive dismantling and rebuilding of areas of cracked exterior wythe stonework and 100% repointing but a careful program of fully grouting the core in order to achieve coherence through the full wall section. As described in the Structural Analysis section successful grouting of a large deep cavity between limestone wythes is a process which is fraught with potential problems. It requires the careful selection and preparation of a gravity flowable solution compatible (slightly weaker) with the strength of the limestone. The efficacy of the cavity filling is difficult to monitor relying on a series of weep holes for checking. On the other hand the fluidity of the grout means that it can potentially flow out any gap so all gaps have to be sealed prior to pouring. This means that all exterior cracks must at least be temporarily filled prior to grouting. With the stability issues of the stone *sans* timber this process would have to be done with the utmost care and planning. The cumulative capital costs of the specialized stone stabilization/restoration process are in stark contrast to those of retaining the timber and shingle. (See below)

Another important consideration in choosing between these major options is the comparative future maintenance and monitoring required by each scenario. The performance and maintenance cycle of the current treatment is predictable, essentially depending on the service life cycle of the shingles and that of the coating applied to them. Exposing the stone on the other hand will require ongoing monitoring and likely, given the exposed, high weathering location much shorter maintenance cycles focused on repointing and timely repair to any cracks. Should there be any period of 'deferred maintenance', typical for buildings where access is an issue, then exponentially increasing deterioration would be the result as it was originally. (Some of the worst problem areas can be ameliorated with subtle flashings but those design improvements would only represent a 'drop in the bucket' were general maintenance to be ignored.)

Lastly in this discussion, it is worth noting that, as viewed during the investigation it was evident that the stone had been historically coated with a lime wash thick enough to obscure the stone coursing, quoins etc. Thus the authentic appearance of the exposed stone walls would likely be as a whitish mass rather than as crisply defined coursed stonework.

In consideration of the issues of: historic authenticity; capital cost and operational cost it behooves the consulting team to recommend the continuation of the timber girt/shingle clad scenario. We do this while fully recognizing what an interesting and challenging project the exposed stone restoration scenario would be and the likely interest it would generate (at least for a while) with the general public.

Preliminary Costing for Budget Purposes (Order of Magnitude):

Common to both options:

Restore exposed stonework at base: $35000.00
(includes: replace stone step; Jahn repair of cracks; break out concrete shoulders, investigate, conserve stone, possibly repour concrete,; repoint joints and clean vertical cracks; premachink corners; pinning at cracked quoins and minor dismantle and rebuild)

Renew Sheet metal base: $7500.00
(includes: investigation but assume complete renewal to allow for improved detailing , e.g. - no exposed fasteners etc.)
Note option of restoring shingles to this area as per pre-1980. $10000.00

Metal roofing replacement : $24000.00

Interior stonework conservation allowance: $35000.00

(includes: repointing, minor dismantle and rebuild focused around window areas)

Replacement of rotted timber lintels: $10000.00

Replacement of decayed floor structure: $12000.00

Replacement of decayed/damaged flooring: $17,000.00
(includes stair railing improvements)

Windows
('as is' option – see below for other window opening scenarios): $ 6000.00
(assume 'cleaning up' of existing metal infill panels; sealing between stone and metal; conservation of remaining original upper wood sash and wood casings; painting)

Plaster Conservation/Finishing $35000.00
(includes repair of missing, damaged and/or cracked areas; removal of recent graffiti; conservation of historic graffiti; finishing

Improve Grading $3500.00
(directly around building)

Temporary Works $35000.00
(Includes establishment of construction road into the site sensitive to flora and fauna and reversible)

Total of works common to both options: <u>$220,000.000</u>
Note: Add $2500.00 if shingling to base stone

Note: It is understood that the removal of debris and potentially hazardous waste from the basement is the responsibility of Ontario Parks.

Walling Option A: Maintain timber girt and wood shingle wall cladding:

Allowance for wall planking repairs:	$30,000.00
Complete Renewal of wood shingle cladding: (assumes Blue Label Certigrade Western Red Cedar)	$170,000.00
In shop dipping or painting of shingles with *In situ* touch-up *Note: Coating of shingles is more for historic accuracy and visibility than durability*	$ 25,000.00
Access: (Scaffold and/or lifts)	$70,000.00
Total Walling Option 'A'	$295,000.00

Total of 'all' construction costs Option 'A': $515,000.00

Walling Option B: Stabilize and Expose Original Stone

Dismantle and rebuild:	$400000.00
100% repointing:	$110,000.00
Gravity grouting:	$800,000.00
Access:	$100000.00
Total Walling Option 'B'	$1,410,000.000

Total of all Construction Costs Option 'B': $1,630,000.00

Note: Above figures do not include stainless steel reinforcement associated with seismic retrofit

Koppla should be aware that in association with the above construction costs, typically, in a project to be tendered the following should also be included in the budget planning:

Contingency: 18% of construction costs
General Contractor's Overhead and Profit: 18% - 20%
(Note: Professional fees, also in addition, will be included in a separate submission)

Lantern/Cupola

As noted earlier the restoration of a Lantern to the lighthouse would be historically appropriate regardless of the period treatment (Walling option 'A' or 'B'). Originally comprised of cast iron panels with glazing and a stepped pedestal base its most distinctive feature was its ogee shaped roof culminating in a ball finial (Fig.15). The 1876 Report of William Sherwood, Inspector of Lighthouses, confirms that the lantern was 9' in diameter with glazing comprised of 10 ½" x 13" lights. It also provides information on the nature of illumination in that period.

This treatment could be replicated based on the combination of historic photos, Baird's specs and comparable surviving lanterns. While cast iron could still form the wall panels the roofing would be undertaken in a non-corroding solderable sheet metal as would the cladding of the pedestal. As any 'new' load is a concern as applied to this sensitive structure consideration might be given to the wall panels being cast in aluminum – non-corroding and much lighter than steel/iron. Regardless, the roof joists would be strengthened in this scenario. Obviously the base of the lantern would have to be integrated with the roof cladding in a weatherproof manner.

Assume $**85000.00** for fabrication and installation of cupola shell. Assume $**125000.00** as actual working light.

Other Window Options

The lancet windows were an important, if problematic, component of the original design. As discussed the problem was associated with their size and alignment at each cardinal elevation. However, if carefully designed it would be possible to feature a full lancet window at each floor level (above main), though offset from the floor below and above, culminating with a view out to the lake from the 5th floor. Design would have to compensate for the greater area in which the planking was discontinuous, ensure that the area around the opening was flashed and sealed to be weather–tight and tempered glass used as vandalism has been known to be an issue.

Obviously this is not a true 'period' treatment but does convey something further in regard to the nature f the original design.

Assume **$45000.00** to implement this approach.

A more modest option would be to simply restore lower sash and glazing to the west openings to replace the metal inserts. With tempered glass the damage that led to the installation of the metal panels should not be an issue. This is the historic treatment from post 1894 to the application of the metal panels c.1980. In this scenario (as well as that above) the incorporation of louvres into the sash at both a 2nd and 5th floor opening would greatly improve ventilation through the structure and thereby diminish condensation at the interior surface. Assume **$12000.00**

Implementation

The isolated location of the site partially surrounded by water has the potential to complicate the construction process. This is further compounded by its placement within a Provincial Park, with extensive, family-based summer tourism and the perceived sensitivity of the surrounding flora and fauna. Still it is important that the work to the exterior (and possibly core) of the lighthouse be undertaken in favorable weather – the typical construction season between April and Thanksgiving. This is particularly true of 'Option B' where the sensitivity of the masonry work - grouting etc., make it **critical.** If this cannot be accommodated then the site will have to be 'housed and heated' with major extra costs not included in the above estimates.

It will also be essential to create much better road access to the site and a reasonable staging area for materials and equipment. It should be possible to construct a temporary and completely removable construction road by the beach without damaging important natural heritage elements. The only other possibility would be for the work to be staged from a barge based offshore (this has not been costed) though the optimum would be to have both possibilities 'in play'.

If work can proceed through the conventional construction season it would be possible, though ambitious, to complete Option 'A' in one 'season' though the assumption should be two years. Option 'B', is very unlikely to be able to be completed in one season, (assuming no housing and heating) and two years is the likely minimum.

As the comprehensive work program still remains a future prospect there are some simple but important measures which should be undertaken *asap.* These consist of:

- Sealing all gaps and holes at the metal roof;
- Filling gaps at, and around, the front entrance step;
- Sealing holes at the metal base

Should the work program become substantially delayed then more significant 'stop gap' maintenance approaches would be required such as:

- Coating the roof with a bituminous or elastomeric coating;
- Undertaking localized repointing at the stone base

Note that should the work program be delayed indefinitely some level of more significant intervention would still be required including the above but extending to the replacement of decayed timber lintels and floor structure.