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Od redakcji

Oddajemy w ręce naszych Czytelników jesienny numer „Wiadomości Konserwatorskich" w 2012 roku.

Okres powakacyjny w środowisku konserwatorskim zdominowała Międzynarodowa Konferencja naukowo-problemowa zorganizowana przez Instytut Budownictwa Politechniki Wrocławskiej, Stowarzyszenie Konserwatorów Zabytków oraz Fundację Doliny Pałaców i Ogrodów Kotliny Jeleniogórskiej.

Konferencja ta jest VIII edycją SAHC (Structural Analysis of Historical Constructions), która w tym roku odbyła się w Polsce, we Wrocławskim Centrum Kongresowym przy Hali Stulecia wpisanej na Listę Światowego Dziedzictwa UNESCO (proj. arch. Max Berg). Zgromadziła około 400 uczestników, a także słuchaczy z kilkudziesięciu krajów świata (poza Europą: z Brazylii, Chin, Indii, Iranu, Japonii, Korei Południowej, Meksyku, Peru, Stanów Zjednoczonych, Tajlandii, Turcji, Zjednoczonych Emiratów Arabskich).

Gośćmi konferencji, których obecność podkreśliła jej znaczenie dla międzynarodowego środowiska konserwatorskiego, byli m.in. Bogdan Zdrojewski, Minister Kultury i Dziedzictwa Narodowego; Rafał Dutkiewicz, Prezydent m. Wrocławia; Piotr Żuchowski, Sekretarz Stanu w MKiDN, Generalny Konserwator Zabytków; Maciej Klimczak, Podsekretarz Stanu w Kancelarii Prezydenta RP oraz kardynał Henryk Gulbinowicz, Senior z Archidiecezji Wrocławskiej.

Swoistym osiągnięciem tej Konferencji było wydanie materiałów naukowych w języku angielskim (3 tomy, 345 referatów, około 3000 stron. Kilkanaście wybranych referatów publikujemy w niniejszym numerze WK).

Organizatorom tej konferencji – największej w historii ochrony zabytków w Polsce, obok Konferencji „Karta Krakowska 2000" zorganizowanej 12 lat temu przez profesora Andrzeja Kadłuczkę, prezesa SKZ – pragniemy złożyć najwyższe gratulacje, tym bardziej, że plon naukowy owego spotkania będzie ważący dla rozwoju ochrony zabytków w Polsce i nie tylko.

Prace redakcji WK oraz Zarządy Głównego SKZ w okresie jesiennym koncentrują się wokół rozwoju „Wiadomości Konserwatorskich" jako periodyku naukowego, problemowego oraz informacyjnego, rozwijającego kontakty ze środowiskiem konserwatorskim.

Biorąc pod uwagę, że do redakcji WK napływa coraz więcej wartościowych artykułów naukowych, podejmujemy działania, aby w roku 2013 nasz periodyk był wydawany jako kwartalnik. Zadanie to jest trudne, ponieważ redakcja WK (czasopismo niekomercyjne) nie posiada własnych środków pozabudżetowych.

Pod koniec 2012 roku składamy naszym P.T. Czytelnikom Życzenia Świąteczne oraz pomyślności w Nowym Roku 2013.

From the Editor

We would like to present our Readers with the autumn issue of "Journal of Heritage Conservation" 2012.

The post-holiday period in the conservators' circles was dominated by the International Conference organised by the Institute of Building Engineering of the Wrocław University of Technology, Association of Monument Conservators and the Valley of Palaces and Gardens Foundation of the Jeleniogórska Valley.

The Conference was the 8th edition of the SAHC (Structural Analysis of Historical Constructions), which this year took place in Poland, in the Wrocław Congress Centre by the Millennium Hall which was entered into the UNESCO World Heritage List (designed by arch. Max Berg). It attracted about 400 participants, as well as visitors from countries all over the world (besides Europe, from: Brazil, China, India, Iran, Japan, South Korea, Mexico, Peru, the USA, Thailand, Turkey, the United Arab Emirates).

Conference guests, whose attendance emphasised its significance for the international conservators' environment, were: Bogdan Zdrojewski, Minister for Culture and National Heritage; Rafał Dutkiewicz, President of Wrocław; Piotr Żuchowski, Secretary of State at the MCNH, General Monument Conservator; Maciej Klimczak, Undersecretary of State at the Chancellery of the President of the Republic of Poland, and Henryk Gulbinowicz, Cardinal of the Wrocław Archdiocese.

A particular achievement of that Conference was publishing its scientific materials in English (3 volumes, 345 papers, about 3000 pages. Several selected papers are published in the current issue of JoHC).

We wish to offer our most sincere congratulations to the organisers of the Conference – the largest in the history of monument protection in Poland, besides the "Krakow Charter 2000" Conference organised 12 years ago by professor Andrzej Kadłuczka, president of AMC – the more so, as the scientific results of that meeting will affect the development of monument protection in Poland and other countries.

In autumn, the work of the JoHC editorial board and the Main Board of the AMC focuses on the development of the "Journal of Heritage Conservation" as a scientific, problem and informative periodical, promoting contacts with conservators' environment.

Considering that more and more valuable scientific articles are sent to the JoHC editors, we are making efforts to have our periodical published as a quarterly in 2013. The task is arduous, since the JoHC (non-commercial magazine) editorial board does not have extra-budgetary means at its disposal.

Towards the end of 2012, we would like to wish our Readers a Merry Christmas and the best of luck in the New Year 2013.

Redaktor Naczelny
Editor in Chief

Kazimierz Kuśnierz

Przewodniczący Rady Naukowej
Chairman of Scientific Board

Jerzy Jasieńko

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The studies on reception of architectural problems between neoclassicism and historicism in the art of Nordic countries in the 19th century

Słowa kluczowe: Dania, Norwegia, Szwecja, Finlandia, historia architektury i urbanistyki, klasycyzm, empire, narodowy romantyzm, neogotyk, neorenesans, neobarok, styl ok. 1900

Rozwój architektury w Skandynawii w dziewiętnastym stuleciu przebiegał w ogólnym zarysie podobnie jak w innych państwach europejskich, atoli w krajach nordyckich neoklasycyzm był na tym obszarze szczególnie stylem preferowanym. Polityczne przemiany, które wstrząsnęły fundamentami *ancien régime*'u w czasie efemerycznego cesarstwa zbudowanego przez cesarza Napoleona I, miały marginalne znaczenie dla rozwoju sztuki na obszarze Danii, Szwecji i Norwegii. Pozostawała przeto Skandynawia poza głównym obszarem teatru wojen napoleońskich, aczkolwiek owe brzemienne w skutkach polityczne wydarzenia nie ominęły również monarchii nordyckich. I tak mimo oddzielenia się Finlandii od Szwecji i wcielenia jej do Imperium Rosyjskiego w 1809 roku, rozwiązania unii duńsko-norweskiej w 1814 roku, utworzenia unii Szwecji z Norwegią w latach 1814-1905 – nie doprowadziły do zerwania tradycyjnych związków politycznych, a nawet więcej – wzmocniły trwające niemal od zawsze więzy społeczno-kulturowe. Innymi słowy, dynastyczne i państwowo-terytorialne, a także ustrojowe roszady na kontynencie, ominęły rewolucyjny i bonapartystyczny zamęt, pozostawiając Skandynawię oazą spokoju i stabilizacji. Jednocześnie ów wielowątkowy ferment spowodował, z jednej strony – umocnienie się tendencji neoklasycystycznych, jak np. styl Ludwika XVI, bądź neoklasycyzm rewolucyjny na przełomie XVIII i XIX wieku, z drugiej strony – przyczynił się do pojawienia się nowych filiacji neoklasycystycznych, takich jak:

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Praca dopuszczona do druku po recenzjach

Key words: Denmark, Norway, Sweden, Finland, history of architecture and urban design, classicism, empire, national romanticism, neo-Gothic, neo-Renaissance, neo-Baroque, style around 1900

The development of architecture in Scandinavia in the nineteenth century in general outline followed the same course as in other European countries, however in the Nordic countries neoclassicism was the particularly preferred style. Political changes which shook the foundations of *ancien régime* in the times of the ephemeral empire built by Emperor Napoleon I, were of marginal significance to the development of art in the territories of Denmark, Sweden and Norway. Scandinavia remained outside the main theatre of Napoleonic wars, although those political events fraught with consequences did not completely pass by the Nordic monarchies. Despite the fact that Finland was separated from Sweden and then incorporated into the Russian Empire in 1809, The Denmark – Norway union was dissolved in 1814, and a union between Sweden and Norway was formed in the years 1814-1905 – it did not lead to breaking traditional political alliances, on the contrary – it strengthened the long-lasting social and cultural ties. In other words, dynastic, state, territorial, as well as political reshuffling on the continent, with its revolutionary and Bonapartist chaos, bypassed Scandinavia which remained an oasis of tranquillity and stability. At the same time that multi-level ferment caused, on the one hand – strengthening of the neo-classicist tendencies, such as e.g. Louis XVI style, or revolutionary neoclassicism at the turn of the 18th and 19th centuries, while on the other – contributed to the appearance of new neo-classicist filiations, such as: the art of

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sztuka dyrektoriatu, konsulatu i szczególnie *empire*. Ta, tradycyjna niejako i relatywnie nowa sztuka i architektura, zyskała na znaczeniu i popularności w takich ośrodkach neoklasycyzmu, jak Kopenhaga i Sztokholm, gdzie akademicki klasycyzm otrzymał wiele inspirujących impulsów, czego przykładem kopenhaska szkoła wykreowana przez Christiana Fredrika Hansena i Michaela Gottlieba Birknera Bindesbølla, rzeźbiarza Bertela Thorvaldsena, a w Norwegii – Christiana Heinricha Groscha, w Finlandii – Carla Ludwiga Engela. Ci właśnie architekci i artyści przekształcili centralne ośrodki sztuki Skandynawii w międzynarodowym stylu „greckiego odrodzenia".

Niepodważalną i trwałą po wsze czasy pozycję w Kopenhadze zajmował architekt Christian Fredrik Hansen, senior rodu architektów. To on otrzymał wszystkie duże publiczne zamówienia, zrealizowane w dwóch pierwszych dziesięcioleciach XIX wieku. Jako profesor kopenhaskiej Królewskiej Akademii Sztuki w latach 1808-1835 odcisnął piętno na całej generacji architektów – nie tylko w Danii, ale również w Norwegii. Zaś rodzina Hansenów sięgała wpływami znacząco na architekturę, między innymi, Monachium, Aten i Wiednia.

the French Directoire, the Consulate and especially the *Empire*. That quite traditional and relatively new art and architecture gained popularity and significance in such centres of neoclassicism as Copenhagen and Stockholm, where academic classicism was given many inspiring impulses, an example of which was the Copenhagen school created by Christian Fredrik Hansen and Michael Gottlieb Birkner Bindesbøll, the sculptor Bertel Thorvaldsen, in Norway – by Christian Heinrich Grosch, and in Finland – by Carl Ludwig Engel. Those architects and artists transformed art centres in Scandinavia in the international style of "Greek Revival".

An indisputable and permanent place of eminence in Copenhagen was held by architect Christian Fredrik Hansen, a senior of a family of architects. He received all important public commissions, realised during the first two decades of the 19th century. As a professor at the Copenhagen Royal Academy of Art, in the years 1808-1835, he left his mark on a whole generation of architects – not only in Denmark but also in Norway. The Hansen family significantly influenced architecture in e.g. Munich, Athens and Vienna. In Scandinavia, and particularly

Ryc. 1. Pałac Christianborg, kaplica, wnętrze. Kopenhaga. Ch.F. Hansen, 1803-1840
Fig. 1. Christianborg Palace, chapel, interior. Copenhagen. Ch. F. Hansen, 1803-1840

Ryc. 2. Pałac Królewski. Oslo. H.D.F. Linstow, 1823-1848
Fig. 2. Royal Palace. Oslo. H.D.F. Linstow, 1823-1848

W Skandynawii, a osobliwie w Danii, wielkimi autorytetami byli wówczas Karl Friedrich von Schinkel, Charles Percier i Pompes Funèbres Fontaine. Ich wpływ jest znacząco widoczny w dwóch monumentalno-reprezentacyjnych królewskich rezydencjach w Kopenhadze i w ówczesnej Christianii, czyli obecnym Oslo. W pierwszym z tych pałaców znajduje się siedziba królów duńskich w Christiansborg'u, wzniesiona przez Christiana Fredrika Hansena w latach 1803-1840, w czystym francuskim *empire à la* Percier i Fontaine. Do tradycji duńskich nawiązywał jedynie wystrój malarski reprezentacyjnych pomieszczeń, które udekorowano dużymi obrazami, pędzla Christophera Wilhelma Eckerberga. Te informacje należy jednak uzupełnić wiedzą, iż poprzedni, oryginalny pałac, zbudowany został *in situ* w stylu barokowym podług projektu Eliasa Davida Häussera między rokiem 1733 a 1745. Po kompletnej destrukcji powstało właśnie owe dzieło Ch.F. Hansena, które uległo zniszczeniu w pożarze w 1884 roku. Kolejna odbudowa pałacu została przeprowadzona przez Eliasa Davida Jørgensena w 1907-1928 w stylu neobarokowym. Rzec można, zataczając niejako dziejowe koło, w którym dostrzega się dziewiętnastowieczną dychotomię, a zarazem *iunctim* między barokiem i klasycyzmem. Z dzieła Ch.F. Hansena ostała się ostatecznie imponująca kaplica pałacowa (Ryc. 1).

Drugim z pałaców zaprojektowanych przez Hansa Ditleva Franciscusa Linstowa (1823-1848) w stylu I cesarstwa jest królewska rezydencja ulokowana na wzgórzu Bellevuehøyden w Oslo. H.D.F. Linstow współpracował przy budowie pałacu z Heinrichem Ernstem Schirmerem z Lipska, któremu należy przypisać schinklowskie inspiracje, jak i wpływy tradycji pompejańskiej, np. berliński Schauspielhaus (1818-1821) – widoczne w sali balowej pałacu. Wnętrza pałacu utrzymane były w niemal czystym stylu empirowo-klasycystycznym, w których wszakże nie zabrakło, podobnie jak w pałacu w Christiansborgu, wymownych akcentów regionalno-narodowych i wernakularnych zarazem, jak choćby w tzw. „ptasim pokoju", urządzonym w latach 1839-1841. Ściany i sufit zdobiły ornamenty pnących roślin, wśród których roiło się od różnego gatunku ptaków, a w tle rozciągały się rodzime krajobrazy, przepojone symboliką zaczerpniętą z nordyckich legend, przy czym dostrzec można w tym salonie wpływy neorokoka, czy, może lepiej – biedermeieru, który był skromniejszą, bardziej *Gemütlichkeit*, niejako mieszczańską wersją *empire.* Również sala audiencjonalna, zwana także „pokojem arkadowym", uznawana jest za przykład stylu I cesarstwa, atoli można także uznać to pomieszczenie jako przykład wczesnego *Rundbogenstil*. (Ryc. 2, 3, 4, 5) Te wnętrza antycypujące eklektyzm nie budzą zdziwienia, bo sam pałac w Oslo był wszak w rzeczy samej czymś w rodzaju eklektycznego neoklasycyzmu.

Jak już powiedziano, kraje nordyckie są, jak pisze David Watkin, „…ważne w historii późnego neoklasycyzmu ze względu na ich wspaniałą architekturę miejską i urbanistykę, chociaż – jak i inne kraje europejskie – przejęły one później wiele cech *Rundbogenstil* i gotyckich, czasami pod wpływem idei tzw. narodowego romantyzmu. (…) Ch.F. Hansen przekształcił Kopenhagę z miasta średniowiecznego i barokowego w neoklasyczne. Jego zespół budowli obejmujący ratusz, sąd i więzienie (1803-1816), połączył dwoma śmiałymi łukami przerzuconymi przez boczną ulicę, jest niezapomnianym opracowaniem w stylu francusko-pruskim, który Friedrich Gilly wywiódł od (Claude-Nicolas – dopis. Z. i T.T.) Ledoux. Hansen w latach 1811-1829 przebudował także katedrę Vor Frue Kirke (NMP). Zaprojektowana w latach 1808-1810, ma

Ryc. 3. Pałac Królewski. Sala balowa. Oslo. H.D.F. Linstow, 1823-1848
Fig. 3. Royal Palace. Ballroom. Oslo. H.D.F. Linstow, 1823-1848

Ryc. 4. Pałac Królewski, tzw. „ptasi pokój". Oslo. H.D.F. Linstow, 1823-1848
Fig. 4. Royal Palace, so called "Bird room". Oslo. H.D.F. Linstow, 1823-1848

in Denmark, great authorities were then Karl Friedrich von Schinkel, Charles Percier and Pompes Funèbres Fontaine. Their impact is clearly visible in two monumental formal royal residences in Copenhagen and Christiania which is today's Oslo. The first of those palaces housed the seat of the Danish kings in Christiansborg, erected by Christian Fredrik Hansen in the years 1803-1840, in pure French *empire à la* Percier and Fontaine. The Danish traditions were alluded to only in the painting decoration of the state rooms which were decorated with huge paintings by Christopher Wilhelm Eckerberg. This knowledge, however, has to be supplemented with information that the former, original palace, was built *in situ* in the Baroque style according to the design by Elias David Häusser between 1733 and 1745. After its total destruction, the work by Ch. F. Hansen was created which was subsequently destroyed in the fire in 1884. The following rebuilding of the palace was carried out in the neo-Baroque style by Elias David Jørgensen in 1907-1928 thus, one could say, completing a circle of history, in which one can see the nineteenth-century dichotomy and, ona szlachetne wnętrze z motywami „doryckimi", o sklepieniu kolebkowym, przypominające projekt Bibliothèque Nationale (Étienne-Louis – dopis. Z. i T.T.) Boulléego.

Uczeń Hansena, Michael Gottlieb Bindesbøll, stworzył w Kopenhadze jedną z najznakomitszych budowli tamtych czasów w Europie – Muzeum Thorvaldsena (położonego nieopodal pałacu Christiansborg – dopis. Z. i T.T.). Przekształcenie istniejącej struktury nastąpiło w latach 1840-1844, chociaż Bindesbøll sporządził projekty już w roku 1834. Muzeum zostało zbudowane by pomieścić rzeźby i zbiory neoklasycznego rzeźbiarza Bartela Thorvaldsena (1768-1844), który podarował je ojczystemu krajowi chociaż przez 41 lat, od roku 1798, mieszkał i pracował w Rzymie. Jego chłodne neoantyczne rzeźby rywalizują z rzeźbami (Antonio – dopis. Z. i T.T.) Canovy w wyrażaniu ideałów (Johanna Joachima – dopis. Z. i T.T.) Winckelmanna, ale muzeum jest zdumiewającym pomnikiem odkrycia polichromii, które położyło kres winckelmannowskiemu wizerunkowi Grecji. Muzeum, zbudowane wokół dziedzińca z klasycznymi fasadami w surowym schinklowskim stylu, z rozglifionymi egipskimi framugami drzwi, szczyci się pełnymi życia przedstawieniami ściennymi wykonanymi w technice tynkowej intarsji na ścianach zewnętrznych. (…) Muzeum Thorvaldsena jest bogato zdobione barwną dekoracją, a na dziedzińcu z nagrobkiem rzeźbiarza ściany jakby zniknęły pod ściennymi przedstawieniami dębów, palm i drzew laurowych. Tak samo malowana natura wkracza w architekturę Bibliothèque Nationale (Henri – dopis. Z. i T.T.) Labrouste'a i przedsionka jego Bibliothèque Saint-Geneviève.

Pod koniec życia Bindesbøll przygotował projekty biblioteki uniwersyteckiej w Kopenhadze, potraktowanej jako szereg pawilonów ze szklanymi dachami wokół kopuły. Jednak zlecenie dano jego uczniowi, Johanowi Davidowi Herholdtowi (1818-1902). Wzniesienie w latach 1855-1861 w typie *Rundbogenstil* tej ceglanej biblioteki stało się przełomowym wydarzeniem w rozwoju romantyczno-narodowej tradycji w architekturze duńskiej drugiej połowy XIX wieku" (Ryc. 6).

Powracając jeszcze na moment do architektury Muzeum Thorvaldsena, warto zauważyć, że obiekt ten był raczej wyjątkowym w obliczu antyczno-greckiej twórczości Thorvaldsena, którą upamiętnił Bindesbøll, wznosząc budynek o wydatnych znamionach francuskiego *empire*, zwanego w Anglii stylem regencji (za panowania regenta, a później króla Jerzego IV). Z kolei inny nieco charakter posiadały cechy norweskiego neoklasycyzmu. W Norwegii, która odseparowała się od Danii i utworzyła unię personalną ze Szwecją w 1814 roku, założenie nowej stolicy w Christianii (Oslo) umożliwiło uczniowi Ch.F. Hansena, Christianowi Henrikowi Groschowi, zaprojektowanie trzech okazałych budowli w stylu „greckiego odrodzenia": Giełdy (1826-1852), Banku Norweskiego (1828) i Uniwersytetu (1841-1852). Ten wybitny twórca norweskiej architektury zbudował znakomity neoklasycystyczny kompleks uniwersytecki, stanowiący świadectwo rozwoju kulturalno-naukowego stolicy, budzącego się do samodzielnego bytu narodu. Gmach Wszechnicy w Christianii (od 1925 roku – Oslo) zaprojektował Ch.H. Grosch, opierając swą koncepcję na wzorach zaczerpniętych z berlińskich neoklasycystycznych budowli K. von Schinkla. Gmach główny Uniwersytetu, założonego w 1811 roku, noszącego miano Universitas Regia Fredericiana powstał jako odpowiednik berlińskiego Uniwersytetu, erygowanego w 1809 roku przez Fryderyka Wilhelma III, za sprawą reform przeprowadzonych przez Karla barona vom Stein i Karla

at the same time, *iunctim* between Baroque and classicism. Ultimately, only the impressive palace chapel remained from the work by Ch. F. Hansen (Fig. 1).

The second palace designed by Hans Ditlev Franciscus Linstow, (1823-1848), in the style of the 1st Empire, was the royal residence located on the Bellevuehøyden hill in Oslo. On the construction of the palace H.D.F. Linstow cooperated with Heinrich Ernst Schirmer from Leipzig to whom the Schinkel inspirations as well as the influences of the Pompeian tradition, e.g. the Schauspielhaus in Berlin (1818-1821), should be ascribed – visible in the palace ballroom. Palace interiors were kept in the almost pure empire-classicist style which, however, like the palace in Christiansborg, did not lack meaningful regional-national and vernacular features, such as those e.g. in the so called "bird room" furnished in the years 1839-1841. The walls and ceiling were decorated with motifs of climbing plants teeming with various species of birds, and with native landscapes, pervaded with symbolism taken from Nordic legends, stretching in the background; at the same time, in this salon it was possible to discern the influence of neo-rococo, or even better – Biedermeier, which was a humbler, more *Gemütlichkeit*, almost bourgeois version of *empire.* The audience chamber, known as the "arcade room", is also acknowledged as the example of the 1st Empire style, although the room could be regarded as an example of the early *Rundbogenstil,* too (Fig. 2, 3, 4, 5). Those interiors anticipating eclecticism do not arouse surprise, since the palace in Oslo itself was indeed a kind of eclectic neo-classicism.

It has already been said that Nordic countries, as David Watkin has written, "…are important for the history of the late neo-classicism because of their magnificent urban architecture and urban planning, although – like other European countries – they later adopted many features of the *Rundbogenstil* and Gothic, sometimes influenced by the idea of the so called national romanticism. (…) Ch.F. Hansen transformed Copenhagen from a medieval and Baroque city into the neo-classicist one. His complex of buildings encompassing the town hall, court of law and jail (1803-1816), linked by two daring arches spanning a side street, is an unforgettable study in the French-Prussian style, which Friedrich Gilly derived from (Claude-Nicolas – add. by Z. and T.T.) Ledoux. In the years 1811-1829, Hansen also altered the Vor Frue Kirke (Our Lady) Cathedral. Designed in the years 1808-1810, it boasts a noble interior with "Doric" motifs and barrel vault, resembling the project of Bibliothèque Nationale by (Étienne-Louis – add. by Z. and T.T.) Boullée.

A disciple of Hansen, Michael Gottlieb Bindesbøll, created in Copenhagen one of the most magnificent edifices of the times in Europe – the Thorvaldsen Museum (located nearby the Christiansborg Palace – add. by Z. and T.T.). The existing structure was transformed in the years 1840-1844, although Bindesbøll had designed it already in 1834. The Museum was built to house the sculptures and collections of the neo-classicist sculptor Bertel Thorvaldsen (1768-1844), who donated them to his homeland even though for 41 years, since 1798, he lived and worked in Rome. His cool neo-antique sculptures rival the sculptures by (Antonio – add. by Z. and T.T.) Canova in expressing the ideals of (Johann Joachim – add. by Z. and T.T.) Winckelmann, but the museum is an amazing monument to the discovery of polychrome which put an end to Winckelmann image of Greece. The Museum, built around a courtyard with classical facades in the austere księcia von Hardenberg, która to Uczelnia przyjęła obecnie nazwę Uniwersytetu Wilhelma i Aleksandra von Humboldtów. Nie bez przyczyny przeto, główna aula w formie antycznego amfiteatru, norweskiej Alma Matris, zaprojektowana przez Ch.H. Groscha, zdradza niewątpliwy wpływ von Schinkla, a centralna klatka schodowa, choć skromniejsza w wymiarach, została skopiowana z berlińskiego Starego Muzeum (*Altes Museum*) z lat 1824-1828 (Ryc. 7).

Z kolei w samej Finlandii sytuacja w sztuce i w architekturze nordyckiej przedstawiała się nieco inaczej, po odłączeniu od Szwecji w 1809 roku i włączeniu do Imperium Rosyjskiego, gdzie stworzono Wielkie Księstwo, posiadające szeroką autonomię trwającą do 1917 roku, z nową stolicą w Helsinkach (Helsingfors). Johan Albrecht Ehrenström sporządził urbanistyczny plan zabudowy centrum miasta, zaś Carl Ludwig Engel, berliński architekt i uczeń von Schinkla, zaprojektował główne budynki publiczne oraz wiele domów prywatnych. Lwia część realizacji C.L. Engla powstała w manierze neoklasycyzmu przenikniętego wpływami rosyjskiego *empire*'u (Adrian Dimitrijewicz Zacharow, Andrej Nikiforowicz Worochin, Thomas de Thomon, Auguste Ricard de Montferrand, Carl I. Rossi), położonych o rzut kamieniem od Petersburga. Helsiński Plac Senacki to jeden z najwspanialszych zespołów europejskiego neoklasycyzmu. Zdominowany został przez wysoką, nakrytą kopułą katedrę luterańską, powstałą w latach 1818-1851. Katedrę flankują: Senat (1818-1822) i Uniwersytet (1828-1832) – obie budowle zdobią monumentalne grecko-doryckie klatki schodowe. Pobliska Biblioteka Uniwersytecka (1833-1845), również C.L. Engla, posiada kopułę z ośmiobocznymi kasetonami i wspaniałe kolumnowe czytelnie (Ryc. 8, 9).

Natomiast w Szwecji, po wygaśnięciu pradawnej dynastii Wazów, w 1811 roku obrano regentem marszałka Francji Jeana Baptiste Bernadotte'a, za którego przyczyną, między innymi, styl I cesarstwa zyskał na trwałej, jak się okazało, popularności, nie tylko wśród sfer dworskich, ale i szerokiej publiczności. Wraz z Bernadotte'em, który koronowany został w 1818 roku wstępując na tron pod imieniem Karola XIV Jana, szwedzka mutacja stylu *empire* określana została jako *Karl-Johann-Stil*. Najwybitniejszym przedstawicielem szwedzkiego *empire*, zwanego także stylem Karola Jana, był architekt Frederik Blom, który jako profesor wykładał w sztokholmskiej Akademii Sztuk Pięknych. Dziełem Bloma jest Rosendal, pałacyk na wyspie Djurgården w centrum Sztokholmu, zbudowany w latach 1823-1830. Jego wyposażenie utrzymane jest w stylu francuskiego *empire*. Bogato zdobione wnętrza wykonali szwedzcy artyści, jednak wyposażenie, brązy i tekstylia zostały sprowadzone z Francji. W głównym pomieszczeniu pałacu, Czerwonym Salonie, ściany pokryte zostały jedwabiem, udekorowane wąskimi wypukłymi kolumnami z mahoniu i zwieńczone fryzem malowanym w technice *en grisaille* na złotym tle. O ile sam fryz nawiązuje w formie do wzorców antycznych, o tyle w treści odwołuje się do miejscowej tradycji. Przedstawiono tam staronordyckich bogów z Odynem na czele, walczących z olbrzymami. Sięganie do motywów z nordyckiej mitologii korespondujące z dążeniami do samookreślenia narodowego, stało się w następnych latach częstą praktyką w sztuce nie tylko w Szwecji, ale również w pozostałych krajach skandynawskich.(Ryc. 10).

Następna generacja architektów również pozostawała w ścisłym związku z kontynentem europejskim. W Szwecji Axel Nyström zastąpił w 1836 roku Frederika Bloma na stanowisku profesora budownictwa, w Danii, po Christianie Frederiku Hansenie, profesorem architektury w 1835 roku

Schinkel style, with splayed Egyptian door frames, boasts of vivid wall representations made using the technique of plaster intarsia on the outer walls. (…) The Thorvaldsen Museum is rich in colourful decorations, and in the courtyard with the sculptor's tombstone the walls seem to have vanished beneath the wall depictions of oaks, palms and laurel trees. In the same way painted nature encroaches upon architecture of the Bibliothèque Nationale by (Henri – add. by Z. and T.T.) Labrouste and the vestibule of his Bibliothèque Saint-Geneviève.

Towards the end of his life Bindesbøll prepared design of the university library in Copenhagen, treated as a row of pavilions with glass roofs around a dome. However, it was his disciple, Johan David Herholdt (1818-1902), who got the commission. Construction of this *Rundbogenstil* – type brick library, in the years 1855-1861, became a breakthrough in the development of the romantic-national tradition in the Danish architecture of the second half of the 19th century" (Fig. 6).

To return for a moment to the architecture of the Thorvaldsen Museum, it is worth noticing that the object was rather unique when compared to the antique-Greek works of Thorvaldsen commemorated by Bindesbøll who erected a building with significant features of the French *empire*, known in England as the regency style (during the reign of the regent, and later King George IV). On the other hand, slightly different features were characteristic for Norwegian neo-classicism. In Norway, which separated from Denmark and formed a personal union with Sweden in 1814, founding a new capital in Christiania (Oslo) enabled a disciple of Ch. F. Hansen, Christian Henrik Grosch to design three impressive buildings in the style of "Greek Revival": the Exchange (1826-1852), the Bank of Norway (1828) and the University (1841-1852). That eminent creator of Norwegian architecture built a magnificent neo-classicist university complex which bore witness to the cultural and scientific development of the capital of the nation awakening to its independent existence. Ch.H. Grosch designed the University Building in Christiania (since 1925 – Oslo) basing his concept on the models found in the neo-classicist buildings by K. von Schinkel in Berlin. The main building of the University, founded in 1811 and known as Universitas Regia Fredericiana, was created as the equivalent of the University in Berlin, founded in 1809 by Frederic Wilhelm III, as a result of reform carried out by Karl baron vom Stein and Karl Duke von Hardenberg, although nowadays the latter is known as Wilhelm and Alexander von Humboldt University. Therefore, not without reason does the main auditorium of the Norwegian Alma Matris, designed by Ch.H. Grosch in the form of an antique amphitheatre, bear unquestionable traces of von Schinkel's influence, and the central stairwell, though more modest in its size, was copied from the Old Museum (*Altes Museum*) in Berlin from the years 1824-1828 (Fig. 7).

On the other hand, in Finland the situation in the Nordic art and architecture was slightly different after the country was separated from Sweden in 1809, and incorporated into the Russian Empire where a Grand Duchy with a new capital in Helsinki (Helsingfors) was established and given much autonomy which lasted until 1917. Johan Albrecht Ehrenström prepared an urban development plan of the city centre, while Carl Ludwig Engel, an architect from Berlin and von Schinkel's disciple, designed the main public utility buildings and numerous private houses. The majority of C.L. Engel's realizations were created in the neo-classicist manner imbued with the został, urodzony w Stuttgarcie, Gustav Frederik Hetsch. Zarówno Nyström, jak i Hetsch byli uczniami Louisa-Hippolyta Lebasa w Paryżu, gdzie Hetsch pracował także u Ch. Perciera i Jeana-Baptiste Rondeleta. Mimo ściśle klasycystycznego wykształcenia zarówno Nyströma, jak i Hetscha cechowała predylekcja ku charakterystycznej dla historyzmu otwartości na kontynuowanie innych stylów z dawnych epok, w tym szczególnie neogotyku, który stosowali z uwagi na skłonność do kultywowania artystycznej i narodowej tradycji. Jako architekci wnętrz byli eklektykami, chociaż wzorem dla nich pozostawali: Precier, Fontaine oraz von Schinkel. G.F. Hetsch, który jako współpracownik Ch.F. Hansena przez wiele lat zajmował się projektowaniem wnętrz pałacu Christiansborg, przełożył na język duński wstęp do *Recueil de décorations intérieures* Ch. Perciera i P.F. Fontaine'a i opublikował je pierwszy raz w 1848 roku, a drugi w 1863.

Dziewiętnastowieczna dyskusja, jaka przetoczyła się niemal we wszystkich ośrodkach sztuki i architektury, a dotycząca chłodnego, winckelmanowskiego ideału antyku, również pozostawiła trwałe ślady na obszarze kultury skandynawskiej, a nade wszystko w duńskiej architekturze i zdobnictwie. Przeto nic dziwnego, że dzieło Gottfrieda Sempera: *Vorläufige Bemerkungen über bemalte Architektur und Plastik bei den Alten*, które ukazało się w 1834 roku w Altonie (ob. Hamburg-Altona), w duńskim wówczas księstwie Szlezwik-Holsztyn, zyskało natychmiast szeroki oddźwięk w duńskiej stolicy, gdzie było recenzowane i komentowane, podobnie jak zaprojektowany i wzniesiony przez G. Sempera w 1833 roku, na zamówienie kupca i bankiera Conrada Hinricha Donnera, gmach Muzeum w Altonie wraz z bogatą polichromią. Sam B. Thorvaldsen miał pochlebnie wyrażać się o polichromowanych pawilonach, w których wystawiano jego rzeźby.

Mało gdzie w Europie przełożono teoretyczne rozważania o polichromii na praktykę z taką odwagą i konsekwencją jak w przypadku Muzeum Thorvaldsena w Kopenhadze, które zostało wzniesione przez architekta Michaela Gottlieba Bindesbølla. Polichromie zastosowano tu zarówno wewnątrz, jak i na zewnątrz. Budynek w kolorze ochry z biegnącym wokół fryzem, usytuowany w pobliżu neoklasycystycznego pałacu Christiansborg, projektu Ch.F. Hansena, wyglądał wręcz prowokująco. Wewnątrz ważnym elementem wystroju był kolor. Głębokie pompejańskie odcienie ścian, kolorowe geometryczne wzory mozaikowych podłóg oraz sklepienia kolebkowe z pompejańskimi motywami tworzyły idealne ramy dla białych rzeźb z marmuru. Całe wyposażenie – gabloty, szafy, stoły i krzesła – zaprojektował architekt, opierając się na wzorach antycznych, szczególnie pompejańskich. Meble były zaskakująco proste, o powierzchniach pozbawionych złoceń, politury czy innych zdobień. Rozwiązania przyjęte przez M.G. Bindesbølla uderzająco wyprzedzają *genre* stosowany w duchu uproszczonego lub inaczej, zredukowanego, neoklasycyzmu w latach dwudziestych XX wieku, czego przykładem może być analogia z głównym hallem gmachu Automobilgesellschaftwerke w Düsseldorfie, autorstwa Petera Behrensa z 1915 roku, bądź inne tego rodzaju awantaże przejęte przez takich mistrzów architektury wnętrz, jak np. Charles Rennie Mackintosh lub też artystów z kręgu Wiener Werkstätte (Ryc. 11).

Polichromia znalazła zastosowanie również w mniej oficjalnych wnętrzach. Wyróżnia się tu szczególnie mieszkanie rzeźbiarza Hermanna Ernsta Freunda przy Frederiksholms-Kanal, niedaleko Muzeum Thorvaldsena. Jego wnętrze, stworzone w latach 1833-1835, wywołało w Kopenhadze duże

influences of Russian *Empire* (Adrian Dimitrievich Zacharov, Andrej Nikiforovich Worochin, Thomas de Thomon, Auguste Ricard de Montferrand, Carl I. Rossi), located a stone's throw from Petersburg. The Senate Square in Helsinki is one of the most splendid neo-classicist complexes in Europe. It is dominated over by the tall Lutheran Cathedral covered with a dome, built in the years 1818-1851. The Cathedral is flanked by: the Senate (1818-1822) and the University (1828-1832) – both buildings are decorated with monumental Greek-Doric stairwells. The nearby University Library (1833-1845), also designed by C.L. Engel, has a dome with octagonal coffers and imposing colonnaded reading rooms (Fig. 8, 9).

In Sweden, after the ancient Vasa dynasty died out, the Marshal of France Jean Baptiste Bernadotte was elected regent in 1811, as a result of which the style of the 1st Empire gained in long-lasting popularity not only among the court circles, but also the Swedish public. Together with Bernadotte, who was crowned in 1818 and ascended the throne as King Karl XIV Johann, the Swedish mutation of the *empire* style was named *Karl-Johann-Stil*. The most eminent representative of the Swedish *empire*, known also as Karl Johann style, was architect Frederik Blom who was a professor at the Stockholm Academy of Fine Arts. A work of Blom is Rosendal, a palace on the isle of Djurgården in the centre of Stockholm, built in the years 1823-1830. Its furnishing is maintained in the style of French *empire*. Richly ornamented interiors were executed by Swedish artists, but the furniture, bronzes and textiles were imported from France. In the main room of the palace, the Red Salon, the walls were covered with silk, decorated with narrow convex columns made of mahogany and topped with a frieze painted in the *en grisaille* technique against golden background. Even though the frieze itself alludes in its form to antique models, its content alludes to local tradition since it depicts Old Nordic deities, with Odin at the head, struggling with giants. Using motifs from Nordic mythology corresponding with aspirations to national self-determination became in later years a frequent habit in art not only in Sweden, but also in other Scandinavian countries (Fig. 10).

The next generation of architects also maintained close ties with the European continent. In Sweden Axel Nyström replaced Frederik Blom at the post of the professor of construction in 1836; in Denmark after Christian Frederik Hansen, Stuttgart-born Gustav Frederik Hetsch became professor of architecture in 1835. Both Nyström and Hetsch were disciples of Louis-Hippolyt Lebas in Paris, where Hetsch worked also for Ch. Percier and Jean-Baptiste Rondelet. Despite strictly classicist education, both Nyström and Hetsch were characterised by a predilection for an openness, so specific for historicism, to continue other styles from previous epochs, particularly neo-Gothic which they used because of a tendency to cultivate artistic and national tradition. As interior designers they were eclectic, even though their ideals were: Precier, Fontaine, and von Schinkel. G.F. Hetsch who, as a co-worker of Ch.F. Hansen, for many years designed the interiors in the Christiansborg palace, translated into Danish the introduction to *Recueil de décorations intérieures* by Ch. Percier and P.F. Fontaine, and published it for the first time in 1848, and for the second in 1863.

The nineteenth-century discussion which took place in almost all art and architecture centres concerning the cool, Winckelmann ideal of the antiquity also left permanent traces in the area of Scandinavian art, and above all in Danish

Ryc. 5. Pałac Królewski. Sala audiencjonalna. Oslo. H.D.F. Linstow, 1823-1848
Fig. 5. Royal Palace. Audience Room. Oslo. H.D.F. Linstow, 1823-1848

Ryc. 6. Muzeum Thorvaldsena. Kopenhaga. M.G. Bindesbøll, 1840-1844
Fig. 6. Thorvaldsen Museum. Copenhagen. M.G. Bindesbøll, 1840-1844

Ryc. 7. Uniwersytet. Oslo. Ch.H. Grosch, 1841-1852
Fig. 7. The University. Oslo. Ch.H. Grosch, 1841-1852

Ryc. 8. Senat. Helsinki. C.L. Engel, 1818-1822
Fig. 8. The Senate. Helsinki. C.L. Engel, 1818-1822

Ryc. 9. Uniwersytet. Helsinki. C.L. Engel, 1828-1832
Fig. 9. The University. Helsinki. C.L. Engel, 1828-1832

Ryc. 10. Pałac Rosendal. Sztokholm. F. Blom, 1823-1830
Fig.10. Rosendal Palace. Stockholm. F. Blom, 1823-1830

Ryc. 11. Muzeum Thorvaldsena, hall główny. Kopenhaga. M.G. Bindesbøll, 1840-1844
Fig.11. Thorvaldsen Museum, main hall. Copenhagen. M.G. Bindesbøll, 1840-1844

poruszenie. Mieszkanie Freunda było jednorodne, stanowiące do pewnego stopnia rodzaj stylowego *Gesamtkunstwerk* (specjalności sztuki i architektury historyzmu), w którym ściany, meble i pozostałe wyposażenie tworzą nierozerwalną jedność. Na ścianach rozciągnięto grubo tkane płótno, pomalowane powyżej czarnego cokołu na cynobrową czerwień, ochrę lub zieleń. Na poszczególnych polach, wydzielonych przez tyrsy i arabeski, umieszczono motywy z dzieła Wilhelma Zahna *Die schönsten Ornamente und merkwürdigsten Gemälde aus Pompeji, Herculanum und Stabiae…,* wydanego w 1828 roku w Berlinie. Drewniane podłogi pomalowane w imitację mozaiki. Meble i inne sprzęty domowe, zaprojektowane przez samego rzeźbiarza, wykonane były z mahoniu lub pomalowane w motywy zaczerpnięte z greckiego malarstwa wazowego. Dekoracje w mieszkaniu Freunda wykonał malarz Georg Christian Hilker, który notabene wykonał również zdobienia w Muzeum Thorvaldsena. Później był zatrudniony jako specjalista od pompejańskiego zdobnictwa w całej niemal Kopenhadze.

Respekt i sentyment dla kultury starożytnej i nowożytnej Italii, podziw dla sztuki Pompejów i Herkulanum był udziałem całego niemal kontynentu europejskiego, nie wyłączając krajów skandynawskich. Atoli nie cała sztuka i architektura krajów nordyckich poddawała się w zupełności wpływom antyku, a osobliwie *empire*. Do niektórych z przejawów przełamania preponderancji neoklasycyzmu i jednocześnie ponownego historycznego rewiwalizmu, tym razem nie w dawnej sielankowo-oświeceniowej wersji, lecz nowej romantyczno-heroicznej i sentymentalnej odsłonie, przyczynił się w Skandynawii wspomniany już H.E. Freund, uczeń B. Thorvaldsena, który należy do zaiste pierwszych twórców rzeźby postaci z nordyckiej mitologii.

Nie można zatem zapominać, mając na uwadze dzieje sztuki północnej Europy, o punkcie zwrotnym historii dziewiętnastego stulecia, który rozpoczął się w 1789 roku, a zakończył, umownie rzecz biorąc, w 1918 roku. Sednem owego zwrotu było zastąpienie monarchistyczno-dynastycznej i kosmopolitycznej wspólnoty, ideą jednorodnego państwa narodowego, która dała początek epoce historyzmu z wszystkimi tego zjawiska konsekwencjami, które obserwowaliśmy jeszcze w XX wieku. Najogólniej rzecz biorąc, istotą historyzmu w wymiarze filozoficznym, teologicznym, politycznym, społecznym była recepcja heglowskiej filozofii historii i sztuki. Innymi słowy, jak konkludują Jacek Purchla i Wolf Tegethoff: „Wiek XIX był w Europie epoką idei narodowej. Jej dyktat zdominował zarówno sztukę, jak i architekturę. W takim kontekście trzeba też widzieć rozwój historii sztuki, ze *Stilgeschichte* (historią stylu) jako najważniejszym osiągnięciem metodologicznym. Relacje historii sztuki i współczesnych nurtów artystycznych opierały się na wzajemnych ustępstwach badaczy i twórców. Debata na temat stylu narodowego osiąga swoje apogeum w ostatniej ćwierci XIX wieku, ale i w następnych dziesięcioleciach pozostaje ważnym czynnikiem ideologizacji…”.

Dla powyższych przyczyn, jak stwierdza Mette Bligaard, ci sami artyści i architekci, którzy tworzyli wnętrza w stylu pompejańskim, zwracali się również ku sztuce średniowiecza. Zainteresowanie różnymi historycznymi stylami tworzyło możliwość wyboru dowolnego stylu – i tak w latach trzydziestych XIX wieku zapanował pod tym względem swoisty pluralizm stylowy. Rezultatem tego zjawiska nie była jednak „bitwa o styl” (*battle of the styles*), lecz raczej współistnienie dwóch tendencji, zarówno wówczas, jak i obecnie, błędnie określanych jako przeciwstawne: europejskiej i narodowej. Dla tej to przyczyny emanacją owego przenikania się i wzajemnych

architecture and ornamentation. Therefore, it is no wonder that the work of Gottfried Semper: *Vorläufige Bemerkungen über bemalte Architektur und Plastik bei den Alten,* published in Altona (present Hamburg-Altona) in 1834, in the then Danish duchy of Schleswig-Holstein, immediately met with a generous response in the Danish capital, where it was reviewed and commented upon, in a similar way to the edifice of the Museum in Altona with its rich polychrome, designed and erected by G. Semper in 1833, commissioned by a merchant and banker Conrad Hinrich Donner. B. Thorvaldsen himself was believed to have favourably commented on polychrome pavilions where his sculptures were displayed.

Hardly anywhere in Europe were theoretical speculations concerning polychrome converted into practice with such courage and consistency as in the case of the Thorvaldsen Museum in Copenhagen, which was erected by the architect Michael Gottlieb Bindesbøll. Polychromes were used here both inside and outside. The ochre-coloured building with a surrounding frieze, located in the vicinity of the neo-classicist Christiansborg palace, designed by Ch.F. Hansen, looked almost provocative. Colour was an important element of its interior decoration. Deep Pompeian hues on the walls, colourful geometric patterns on mosaic floors and barrel vaults with Pompeian motifs, constituted ideal frames for white marble status. All the furniture – cabinets, wardrobes, tables and chairs – were designed by the architect, basing on ancient models, particularly Pompeian. The furniture was surprisingly simple, their surfaces devoid of gilt, French polish or other decoration. Solutions applied by M.G. Bindesbøll were strikingly ahead of the *genre* used in the spirit of the simplified or, in other words, reduced neo-classicism in the 1920s, an example of which could be analogy to the main hall of the Automobilgesellschaftwerke building in Düsseldorf, designed by Peter Behrens in 1915, or other such advantages adopted by such masters of interior design as e.g. Charles Rennie Mackintosh, or artists from the Wiener Werkstätte circles (Fig. 11).

Polychrome found its use also in less official interiors. Particularly noteworthy here were the lodgings of a sculptor Hermann Ernst Freund at Frederiksholms-Kanal, close to the Thorvaldsen Museum. Its interior, created in the years 1833-1835, caused a big stir in Copenhagen. Freund's flat was homogeneous, to a degree constituting a kind of stylistic *Gesamtkunstwerk* (a specialty of art and architecture of historicism), in which walls, furniture and other equipment make up an inseparable unity. The walls were covered with thick-woven canvas, painted vermilion red, ochre or green above a black plinth. In individual panels sectioned off by thyrsi and arabesques, there were placed motifs from the work by Wilhelm Zahn *Die schönsten Ornamente und merkwürdigsten Gemälde aus Pompeji, Herculanum und Stabiae…,* published in 1828 in Berlin. Wooden floors were painted to imitate mosaic. Furniture and other household items, designed by the sculptor himself, were made from mahogany or painted with motifs borrowed from Greek vase painting. Decorations in Freund's apartment were made by painter Georg Christian Hilker, who also executed the ornaments in the Thorvaldsen Museum. Later he was employed as a specialist in Pompeian decoration in almost the whole of Copenhagen.

Respect and sentiment for the culture of the ancient and modern Italy, admirations for the art of Pompeii and Herculaneum was shared all over the European continent, the Scandinavian countries not excepted. Nevertheless, not relacji między europejskim, *alias* międzynarodowym, neoklasycyzmem i jednocześnie narodowym, historyczno-patriotycznym neoantykiem, jest pełna patosu i sentymentalizmu oraz narodowo-mitologicznej chwały świątynia Walhalla, nieopodal Ratyzbony (Regensburga), wzniesiona w latach 1831-1842 przez Leo von Klenzego. Innego rodzaju nastrojową antynomią jest Befreiugshalle koło Kelheim, wybudowana w latach 1847-1863 przez tegoż von Klenzego, który tym samym dołączył do zwolenników popularnego w Niemczech *Rundbogenstil*, pozostając w przekonaniu, że w tej „okrągłołukowej" konstrukcji przetrwały zasady architektury greckiej. I choć nie projektował on w czystym *Spitzbogenstilu*, to w późniejszym okresie starał się łączyć formy greckie z gotyckimi. Jednakże narodowo-patriotyczny romantyzm przepojony uwielbieniem dla antyku, miał również drugie, mniej idealistyczne, a za to bardziej pełne emocjonalnych uniesień oblicze. Drugą, równie znaną postacią, jaką przybierał nie tylko narodowy romantyzm, był historyzm w jego najpopularniejszej odmianie neomediewalizmu, a osobliwie, zaskakującej tajemniczości i niesamowitości architektury neogotyckiej. Ów styl niezmiennie trwający od około 1750 roku, aż do – nawet pierwszych dekad XX wieku, ewoluował w dziewiętnastym stuleciu od około 1800 – mając na myśli epokę romantyzmu, a następnie od około 1860 – mając na uwadze epokę pozytywizmu, do mniej więcej około 1914 roku. Inaczej rzecz ujmując, zarówno na niemieckojęzycznym obszarze, jak i w krajach nordyckich, spotykamy się w Europie z fenomenem paradygmatu, swoistej 'arki przymierza' neoklasycyzmu i historyzmu.

Nie był przeto umiarkowanie znaczącym udział w architekturze skandynawskiej kreacji i manifestacji sublimacji oraz wzniosłości neogotyku, w którym istotną rolę odgrywały, jakżeż ważne dla kultury XIX wieku, pierwiastki emocjonalno--narodowe. Atoli, po bolesnych doświadczeniach dwudziestego stulecia, nie od rzeczy warto przytoczyć inną jakościowo konstatację o historycznej roli narodu w sztuce wieku XIX. Ten problem ujął Wolf Tegethoff w następujących słowach: „… styl narodowy zrodził się jako mit, jako rzekomo historyczny fakt, tak samo nieprawdziwy jak pojęcie narodów przednowoczesnych. Jednak w ideologicznym kontekście nacjonalizmu wieku XIX i początku XX sprawdzał się doskonale jako konstrukcja hipotetyczna. Natomiast w przypadku konieczności zdefiniowania odpowiedniego stylu wyrażającego tożsamość narodową koncepcja ta okazywała się dość śmiesznym uproszczeniem". Ten odważny i nieco ryzykowny pogląd nie zmienia ogólnie przyjętego uznania, że Goci pochodzili ze Skandynawii, a styl neogotycki jest w krajach nordyckich uważany za narodowy. W ten sam sposób postępowali Brytyjczycy, Niemcy, nie wyłączając Francuzów – utrzymując, że to właśnie neogotyk był ich narodową specjalnością. Nie bez kozery przeto, utworzony w 1811 roku Związek Gotycki (*Gotische Liga*) powstał w Szwecji w celu ponownego wprowadzenia legendy Wikingów i sag islandzkich do sztuki północnoeuropejskiej i literatury. Gorącym zwolennikiem i członkiem Związku był następca tronu książę Oskar, syn króla Karola XIV Jana. Nie było rzeczą przypadku, że Karol Jan nadał swojemu synowi imię Oskar, to samo, które otrzymał syn Osjana, legendarnego wojownika i barda celtyckiego, od swojego ojca. Następca tronu dał konkretny wyraz zainteresowaniu 'gotycyzmem' polecając urządzić jeden z apartamentów w pałacu królewskim w Sztokholmie wyłącznie w stylu neogotyckim. Nadmienić wypada, iż sztokholmska rezydencja królów Szwecji posiada imponującą metrykę świadczącą o ciągłości historycznej tego zabytku, który

all the art and architecture of the Nordic lands fully submitted to the influence of the antiquity, and particularly *empire*. In Scandinavia some manifestations of overcoming the preponderance of neo-classicism, and simultaneously of another historical revivalism, though this time not in its former bucolic-enlightenment version but a new romantic-heroic and sentimental rendition, were contributed to by the already mentioned H.E. Freund, a disciple of B. Thorvaldsen, who indeed should be named among the first sculptors of figures from the Nordic mythology.

One should not forget, when considering the history of art in northern Europe, about the turning point in the history of the nineteenth century, which began in 1789 and finished, symbolically, in 1918. The crux of the change was replacing the monarchist-dynastic and cosmopolitan commonwealth with the idea of homogeneous national state, which gave rise to the historicism epoch with all the consequences of that phenomenon which we could still observe in the 20th century. Generally speaking, the essence of historicism in its philosophical, theological, political and social dimension, was reception of Hegel's philosophy of history and art. In other words, as Jacek Purchla and Wolf Tegethoff have concluded: "The 19th century was in Europe the epoch of a national idea which was predominant both in art and architecture. In that context one has to see the development of art history, with *Stilgeschichte* (history of style) as its most vital methodological achievement. Relations between history of art and modern artistic trends were based on mutual concessions of scientists and creators. The debate concerning the national style reached its apogee in the last quarter of the 19th century, but remained an important factor of ideologisation throughout the following decades,…".

For those reasons, according to Mette Bligaard, the same artists and architects who created interiors in the Pompeian style, turned also towards medieval art. Interest in various historic styles offered an opportunity to choose any style – and so in the 1830s a specific stylistic pluralism could be observed. However, this phenomenon did not result in a *battle of the styles* but rather a coexistence of two tendencies, both then and nowadays wrongly interpreter as contradictory: the European and national. For that reason, the reflection of such interpenetration and mutual interrelations between European *alias* international neo-classicism and, at the same time national, historical-patriotic neo-antiquity, is the full of pathos, sentimentalism and national-mythological glory temple of Walhalla situated near Regensburg, erected in the years 1831-1842 by Leo von Klenze. Another type of atmospheric antinomy is Befreiugshalle near Kelheim, built in 1847-1863 by the same von Klenze who thus joined the supporters of the *Rundbogenstil*, popular in Germany, being convinced that such a "rounded-arch" construction preserved the principles of Greek architecture. And although he did not design in the pure *Spitzbogenstil*, at a later period he tried to combine Greek and Gothic forms. However, the national-patriotic romanticism imbued with admiration for the antiquity had also another face, less idealistic, but full of emotional elation. The other, equally well-known form, taken not only by national romanticism, was historicism in its most popular version of neo-medievalism, and especially the surprising mysteriousness and uncanniness of neo-Gothic architecture. That style lasting unvaryingly from around 1750 until even the first decades of the 20th century, evolved in the nineteenth century since pod tym względem przyrównać można do Hradczan, Luwru czy też zamku berlińskiego. Pierwotnie siedziba szwedzkich monarchów powstała *in situ* w 1240 roku jako zamek obronny, który po bardzo licznych przebudowach, uzyskując ostatecznie kształt ogromnego założenia, które przybrało formy w typie baroku rzymskiego. Autorem owej, będącej dumą Szwedów, budowli był Nicodemus Tessin młodszy, uczeń Gianlorenza Berniniego. Po różnych perypetiach dziejowych sam pałac zrealizowany został w latach 1692-1754. Jest rzeczą oczywistą, że w tego rodzaju znakomitym obiekcie nawarstwiały się kolejno świadectwa zmieniających się stylów i mód, w tym również nowych, tym razem dziewiętnastowiecznych trendów, takich jak neogotyk bądź styl II cesarstwa, które odnalazły się we wnętrzach sztokholmskiego królewskiego pałacu.

Wspomniana już wyżej komnata pałacowa w stylu neogotyckim wyłożona została jasnym mazerowanym drewnem brzozowym, gatunkiem uważanym za szczególnie „narodowy" (ludowy), ściany podzielono na pola za pomocą dekoracyjnych maswerków, sufit przypominał gotyckie sklepienie żebrowe. Kompletne umeblowanie zostało wykonane przez zamkowego stolarza według wskazówek księcia. Następca tronu Oskar, który w 1844 roku został królem Oskarem I, wybrał neogotyk również jako styl zameczku Oscarshall, położonego w Bygdøy niedaleko Oslo. Projekt i realizację tego najbardziej znanego w Norwegii przykładu świeckiej architektury neogotyckiej wykonał duński architekt Johan Henrik Nebelong w latach 1847-1852. J.H. Nebelong jednoznacznie inspirował się wykonanym na podstawie koncepcji Karla Friedricha von Schinkla, a zaprojektowanym z zespołem: Ludwig Persius i Johann Heinrich Strack – pałacykiem Schloss Babelsberg koło Poczdamu, wzniesionym między 1834 a 1849. Na ideę zameczku Oskara I nie bez wpływu okazały się takie obiekty, jak np. Burg Stolzenfels koło Koblencji, arch. arch.: Johann Claudius Lassaulx, K.F. von Schinkel, Friedrich August Stüler, 1823-1847. Dla przypomnienia, powstały w podobnym duchu pałacyki-zameczki, jak np. w Opinogórze, Henryk Marconi (atrybowany), 1828-1838, rozbudowany 1843, przebudowany 1894 przez Józefa Hussa; w Kossów-Mereczowszczyznie, Franciszek Jaszczołd, 1838; w Rokosowie, Friedrich August Stüler, 1847-1850 (Ryc. 12, 13, 14, 15).

Wnętrza zameczku Oscarshall określały w największym stopniu założenia ikonograficzne, pochodzące z epoki. Wszędzie podkreślano narodowy charakter obiektu. Norweska przyroda, historia i norweski lud były głównymi tematami obrazów. W boazerię jadalni wprawiono sześć dużych malowideł, które przedstawiały rodzime krajobrazy, zaś nad fryzem przedstawiono sceny z życia rolników. Jeden z salonów ozdobiono podobiznami średniowiecznych królów i orężem, aby przypominały o czasach wielkości Norwegii. W pokojach królewskich ściany nad boazerią pokrywały przedstawienia rodzimej natury i motywy zaczerpnięte z islandzkiej sagi o legendarnym Fridthiofie Śmiałym, a reliefy figuralne z tej sagi tworzą fryz obiegający górą pomieszczenie. W dużej mierze wykonano go z materiałów zastępczych, jak to było w powszechnym zwyczaju w dziewiętnastowiecznej architekturze, używając substytutów w miejsce szlachetnych materiałów, a jednak mimo wszystko sprawia efektowne wrażenie. Malowidła i dekoracje wykonali artyści pochodzący z Norwegii, ktorzy najczęściej pracowali w Düsseldorfie: Hans Gude, Adolf Tidemand i Joachim Christian Frich. Atoli, stałym i jednocześnie podstawowym elementem architektury wnętrz pozostawał styl Tudorów, powszechnie obowiązujący w europejskim budownictwie neogotyckim.

around 1800 – meaning the romantic epoch, and then since around 1860 – meaning the positivistic epoch, until more or less 1914. In other words, both in the German-speaking territories and in the Nordic countries, we encounter in Europe the phenomenon of a paradigm, a specific 'ark of covenant' between neo-classicism and historicism.

Therefore, the participation of creation and manifestation of neo-Gothic sublimation and loftiness with the emotional-national elements so important for the culture of the 19th century, was not moderately significant in Scandinavian architecture. However, after painful experiences of the twentieth century, it might be worthwhile to quote an observation of different quality concerning the historic role of nation in the art of the 19th century. The issue was thus addressed by Wolf Tegethoff: "… the national style was born as a myth, as an allegedly historic fact, as untrue as the notion of pre-modern nations. Nevertheless, in the ideological context of nationalism of the 19th and the beginning of the 20th century it worked perfectly as a hypothetical construction. But when it was necessary to define an appropriate style expressing a national identity the concept turned out to be a ridiculous simplification." That brave and slightly risky opinion does not change the generally accepted belief that Goths came from Scandinavia, and the neo-Gothic style is regarded as national in the Nordic countries. The British, Germans, even the French behaved in the same way – maintaining that neo-Gothic was their national speciality. So not without reason, in 1811 a Gothic League (*Gotische Liga*) was formed in Sweden in order to reintroduce the Viking legend and Icelandic sagas into the north-European art and literature. An ardent supporter and member of the League was Prince Oscar, the heir to the throne, son of King Karl XIV Johann. It was not a coincidence that Karl Johann named his son Oscar, the name given by Ossian, a Celtic legendary warrior and bard, to his son. The heir to the throne clearly expressed his interest in 'gothicism' by having one of apartments in the royal palace in Stockholm furnished exclusively in the neo-Gothic style. It should be mentioned, that the Stockholm residence of kings of Sweden boasts an impressive 'pedigree', the evidence of historic continuity of that monument which, in this respect, can be compared to Hradcany, the Louvre or the castle in Berlin. Originally , the seat of Swedish monarchs was built *in situ* in 1240 as a fortified castle which, after numerous alterations, finally obtained the enormous layout with forms in the type of Roman Baroque. The author of that building, which is the pride of the Swedes, was Nicodemus Tessin the younger, a disciple of Gianlorenzo Bernini. After historical vicissitudes, the palace was realised in the years 1692-1754. It seems obvious, that in such a magnificent object there accumulated evidence of subsequently changing styles and fashions, including the new, this time nineteenth-century trends, such as neo-Gothic or the style of the 2nd Empire, which can be found in the interiors of the royal palace in Stockholm.

The already mentioned palace chamber in the neo-Gothic style was laid with light grained birch wood, a species regarded as particularly "national" (folk), the walls were sectioned off into panels by means of decorative tracery, the ceiling resembled a Gothic rib vault. All the furniture was made by the castle carpenter according to the Prince's instructions. The heir to the throne Oscar who, in 1844, became King Oscar I, also chose neo-Gothic as the style for the Oscarshall castle, situated in Bygdøy near Oslo. The design and realization of this best

Ryc. 12. Zameczek Oscarshall. K. Oslo. J.H. Nebelong, 1847-1852
Fig.12. Oscarshall castle. K. Oslo. J.H. Nebelong, 1847-1852

Ryc. 13. Zameczek Oscarshall, wnętrze. K. Oslo. J.H. Nebelong, 1847-1852
Fig.13. Oscarshall castle, interior. K. Oslo. J.H. Nebelong, 1847-1852

Ryc. 18. Muzeum Zoologiczne, dziedziniec. Kopenhaga. Ch. Hansen, 1863-1869
Fig.18. Zoological Museum, courtyard. Copenhagen. Ch. Hansen, 1863-1869

Ryc. 15. Zameczek Oscarshall, wnętrze. K. Oslo. J.H. Nebelong, 1847-1852
Fig.15. Oscarshall castle. K. Oslo. J.H. Nebelong, 1847-1852

Ryc. 16. Biblioteka Uniwersytecka. Kopenhaga. J.D. Herholdt, 1861-1866
Fig.16. University Library. Copenhagen. J.D. Herholdt, 1861-1866

Ryc. 17. Parlament. Oslo. E.V. Langlet, 1858-1866
Fig.17. Parliament. Oslo. E.V. Langlet, 1858-1866

Ryc. 14. Zameczek Oscarshall, wnętrze. K. Oslo. J.H. Nebelong, 1847-1852
Fig.14. Oscarshall castle, interior. K. Oslo. J.H. Nebelong, 1847-1852

Lata czterdzieste XIX wieku w Danii, Norwegii i Szwecji naznaczone były silnymi wpływami etniczno-kulturowej wspólnoty narodowej. Poczucie jedności między tymi trzema krajami znalazło wyraz w ruchu zwanym „skandynawizmem", podkreślając wspólne elementy językowe, społeczne i historyczne tych państw. W Danii powstało Towarzystwo Skandynawskie, w którym profesor historii sztuki N.J. Høyen nawoływał do rozwijania skandynawskiej sztuki narodowej, uwzględniającej wszakże odrębności regionalne i językowe, harmonizujące z dziedzictwem artystycznym. Równocześnie dawała znać o sobie swego rodzaju opozycja wobec kontynentalnego akademizmu mająca na celu pielęgnację korzeni nordyckiej tradycji. Sięgano przeto szeroko do motywów staronordyckich i sztuki ludowej. Z kolei w Szwecji podobne towarzystwo powstało w 1846 roku. Wyrazem tego, podobnie jak w innych krajach skandynawskich, rosło, trwające do dziś, zainteresowanie architekturą drewnianą symbolizującą, między innymi, przywiązanie do protestanckiego etosu skromności i estetycznego utylitaryzmu. Drewno oznaczało powrót do naturalnych materiałów budowlanych, jak również zastosowanie nietynkowanej cegły, która powróciła do łask w budynkach publicznych. Przykładem podobnych tendencji mogą być dwa gmachy, będące niejako archetypami tego rodzaju modusu budowlano-architektonicznego. Pierwszym utrzymanym w tym stylu jest budynek Biblioteki Uniwersyteckiej w Kopenhadze, wzniesiony przez Johana Daniela Herholdta w latach 1861-1866. Reprezentacyjna klatka schodowa o ścianach z żółtej nietynkowanej cegły przykryta została drewnianym, belkowym, polichromowanym stropem. W czytelni zastosowano po raz pierwszy w Danii konstrukcję z nieobudowanych żeliwnych kolumn. Podobne dzielą wnętrze, przypominające bazylikę, na sklepioną nawę główną i dwie niższe nawy boczne. Konstrukcja i wystrój wewnętrzny przywodzą na myśl, wzorcową pod tym względem, Bibliotekę Saint-Geneviève w Paryżu, Henri Labrouste'a, wybudowaną między 1843 a 1850. Drugim z analogicznych obiektów jest gmach Parlamentu w Oslo. Jest to dzieło szwedzkiego architekta Emila Viktora Langleta, który budynek ten zaprojektował w 1858 roku, lecz ostatecznie pracę zakończył w 1866 roku. Główna sala zgromadzeń zaprojektowana została w postaci wysuniętej do przodu, przed lico obiektu, półokrągłej rotundy. Wnętrze wyłożono drewnem imitującym dąb i udekorowano złotem, galerie i strop zostały wykonane również w drewnie. Architektura wnętrz nawiązuje do stylu Tudorów, jednak stosowane w zdobieniach nordyckie ornamenty roślinne w kolorach czerwonym, białym, niebieskim i popielatym – sprawiło wrażenie, że wystrój uznano za zgodny z narodowym charakterem. Norweski Parlament jest ważną komponentą unikalnej kompozycji urbanistycznej konstytuującą faktycznie 'narodowe forum', w skład którego wchodzą również opisane budynki: Pałacu Królewskiego, Parlamentu, Muzeum Narodowego, Uniwersytetu i Teatru Narodowego. Zważyć warto, że forum w Oslo jest założeniem ostatecznie zamkniętym, w przeciwieństwie do nigdy nieukończonych takich koncepcji urbanistycznych, jak: fryderycjańskie forum w Berlinie, forum cesarskie w Wiedniu, którego podwaliny założone zostały w ostatniej dekadzie panowania Franciszka Józefa I oraz forum cesarskie w Poznaniu, którego budowie patronował Wilhelm II.

Gmachy Parlamentu norweskiego oraz kopenhaskiej Biblioteki Uniwersyteckiej były uznawane wówczas i są nadal obecnie, jak utrzymuje M. Bligaard, za szczególnie udane przykłady narodowej architektury, w których upatruje on silne wpływy lombardzkiego renesansu. Wszelako należy odnotować, że obydwa te obiekty uznać wypada za historyzu-

known example of lay neo-Gothic architecture in Norway was executed by a Danish architect Johan Henrik Nebelong, in the years 1847-1852. J.H. Nebelong was unquestionably inspired by the Schloss Babelsberg, a palace near Potsdam built between 1834 and 1849, on the basis of the concept by Karl Friedrich von Schinkel, and designed with a team of Ludwig Persius and Johann Heinrich Strack. The castle of Oscar I was also influenced by such objects as e.g. Burg Stolzenfels near Koblenz, built by architects: Johann Claudius Lassaulx, K.F. von Schinkel, Friedrich August Stüler, 1823-1847. As a reminder, palaces – castles in the same spirit were erected e.g.: in Opinogóra, Henryk Marconi (attributed), 1828-1838, extended in 1843, altered in 1894 by Józef Huss; in Kossów-Mereczowszczyzna, Franciszek Jaszczołd, 1838; in Rokosow, Friedrich August Stüler, 1847-1850 (Fig. 12, 13, 14, 15).

To the greatest extent, the interiors in the Oscarshall castle were defined by iconographic layout of the epoch. The national character of the object was emphasised everywhere. Norwegian nature, history and Norwegian people were main subjects of pictures. Six large paintings representing native landscapes were fitted into the wood panel ling in the dining room, while above the frieze scenes from the life of farmers were depicted. One salon was decorated with likenesses of medieval kings and with weapons in order to recall the times of Norway's greatness. In royal rooms the walls above wood panelling were covered with images of native nature and motifs borrowed from the Icelandic saga about the legendary Fridthjof the Bold, and figure reliefs from the saga constitute the frieze running around the room at the top of the wall. It was largely made from substitute materials, which was quite common in the nineteenth-century architecture, used in place of more precious materials, nevertheless, it still makes a stunning impression. Paintings and decorations were made by artists from Norway, who most frequently worked in Düsseldorf: Hans Gude, Adolf Tidemand and Joachim Christian Frich. However, the permanent and simultaneously the basic element of interior design remained the Tudor style, commonly applied in European neo-Gothic building.

The 1840s in Denmark, Norway and Sweden were marked by strong influence of the ethnic-cultural national community. The feeling of unity between the three countries was expressed in the movement called "Scandinavism", by emphasising common language, social and historical elements in those countries. In Denmark a Scandinavian Society was established, in which professor of art history N.J. Høyen called for developing Scandinavian national art, though taking into account region al and linguistic differences, in keeping with artistic heritage. At the same time, a kind of opposition against continental academism emerged, aiming at cultivating the roots of Nordic tradition. Thus the Old Norse motifs and folk art were widely used. In Sweden a similar society was formed in 1846. There, like in other Scandinavian countries, its consequence, lasting until today, has been a growing interest in timber architecture symbolising, among other, attachment to the protestant ethos of modesty and esthetic utilitarianism. Timber meant return to natural building materials, as well as the use of unplastered brick which returned into favour in public utility buildings. As examples of similar tendencies can serve two edifices, which were almost archetypes of that building-architectural modus. The first maintained in such a style is the building of the University Library in Copenhagen, erected by Johan Daniel Herholdt in the years 1861-1866. The formal staircase with jący eklektyzm, w którym znać silne wpływy mediewalnego rewiwalizmu z neorenesansem pozostającym w manierze *Rundbogenstil* (Ryc. 16, 17).

Wspomniane już wyżej średniowieczno-gotyckie reminiscencje nie cieszyły się szczególnym wzięciem u twórców architektury skandynawskiej, w zdecydowanym przeciwieństwie do neorenesansu, który pozostawał pod dominującym wpływem budownictwa niemieckiego. Recepcja neorenesansu nordyckiego przyjmowała tamże głównie postać różnych form *Rundbogenstil* oraz neostylowych odmian neorenesansu, a raczej manieryzmu niderlandzkiego bądź flamandzkiego, w których dawało się odczuć oddziaływanie architektury weneckiej. Do znakomitych przykładów północno-włoskiego i północnoeuropejskiego, neorenesansowego manieryzmu należy *Gesamtkunstwerk*, w wydaniu jednego z najwybitniejszych mistrzów dziewiętnastowiecznego historyzmu i klasycyzmu, czyli Christiana Hansena młodszego, brata Theophila Hansena. Nie można przeto ominąć w panoramie architektury kopenhaskiej gmachu Muzeum Zoologicznego, wzniesionego w latach 1863-1869 w stylu *Rundbogenstil* przez Christiana Hansena. Już na pierwszy rzut oka widać dosłownie, że budowla ta, z uwagi na podobieństwo funkcjonalne, przywodzi na pamięć wcześniejszy obiekt wzniesiony przez Theophila Hansena, czyli wiedeńskie Muzeum Broni, będące integralną częścią Arsenału, który powstał między rokiem 1849 a 1856.

Muzeum Zoologiczne Christiana Hansena koresponduje z ideą renesansowo-manierystycznego w typie *palazzo cortile*, z nieodzownym dziedzińcem centralnym, który w przypadku kopenhaskiego pałacu nauk przyrodniczych nakryty został szklaną konstrukcją. Dziedziniec obiektu okalają trzypiętrowe arkady, dolne pozostają utrzymane w stylu neorenesansowym, natomiast druga i trzecia kondygnacja zostały zaprojektowane w stylu neobizantyńskim, który wcześniej wprowadził Theophil Hansen w Muzeum Broni (Ryc. 18).

Przerywając na moment tok chronologii architektury XIX wieku w Skandynawii, nie można pominąć znakomitych zasług obu wspomnianych Duńczyków w dziedzinie rewitalizacji antyku w świeżo odrodzonej, nowoczesnej Grecji. Christian i Theophil Hansenowie ozdobili współczesne centrum Aten wystawiając, tak ważne dla kultury stolicy Królestwa Hellenów, obiekty jak: Uniwersytet (Ch. Hansen, 1839-1850), Akademia Nauk (Th. Hansen, 1859-1887) i Biblioteka Narodowa (Th. Hansen, 1885-1921). Ich surowy styl 'greckiego odrodzenia' nacechowany był krystaliczną wręcz czystością detalu. Zdobyte w Grecji doświadczenia pozwoliły na rozwinięcie pełnych inwencji i imaginacji talentów wzbogacających artystyczną paletę architektury zarówno neoklasycyzmu, jak i eklektycznego historyzmu.

W kręgu neorenesansowego *Rundbogenstil*, uwzględniając również muzealny charakter budynku, pozostaje Muzeum Narodowe w Sztokholmie, które zaprojektował w latach 1844-1866 Friedrich August Stüler, uczeń K. von Schinkla. Budynek sztokholmskiego muzeum, należący do pierwszych tego rodzaju państwowych instytucji publicznych, nosi wyraźne znamiona, szeroko rozumianej środkowoeuropejskiej szkoły neorenesansu, rozciągającej się wpływami od Skandynawii aż po Bałkany. Animatorami tego kierunku zwanego powszechnie jako *Rundbogenstil*, którym był i pozostaje nadal neorenesansowy amalgamat, byli głównie F.A. Stüler, Friedrich von Gärtner, Heinrich Hübsch, Ludwig Persius i Friedrich Bürklein. Sam zaś gmach Muzeum Narodowego w Sztokholmie wyraża, coś na kształt pośredniego *genre*'u F.A. Stülera, zapoczątkowanego

walls of yellow unplastered brick was covered with polychrome timber beam ceiling. In the reading room a construction from non-encased cast-iron columns was used for the first time in Denmark. Similar columns divide the interior resembling a basilica into the vaulted central nave and two lower side aisles. The construction and interior decoration bring to mind, ideal in this respect, Saint-Geneviève Library in Paris by Henri Labrouste, built between 1843 and 1850. The other analogical object is the building of Parliament in Oslo. It is the work of a Swedish architect Emil Viktor Langlet, who designed the building in 1858, but completed the work in 1866. The main assembly room was designed in the form of a semi-circular rotunda protruding from the object facade. The interior was laid with wood imitating oak and decorated in gold; galleries and the ceiling were also made of wood. Interior design alludes to the Tudor style, however, Nordic plant ornaments in red, white, blue and grey colour used in decoration caused the interior décor to be acknowledged as in keeping with the national character. The Norwegian Parliament is a significant component of a unique urban composition constituting in fact 'national forum', encompassing also the described buildings: the Royal Palace, Parliament, the National Museum, the University and the National Theatre. It is worth noticing, that the forum in Oslo is a fully completed layout, as opposed to such never completed urban concepts as: Forum Fridericianum in Berlin, the Imperial Forum in Vienna the foundations of which were laid during the last decade of the reign of Franz Joseph I, and the imperial forum in Poznan the building of which was under the patronage of Wilhelm II.

The buildings of the Norwegian Parliament and of the University Library in Copenhagen have been perceived, according to M. Bligaard, as particularly successful examples of national architecture, in which strong impact of Lombardian Renaissance has been sought. Nevertheless, it should be noted that both objects ought to be regarded as examples of historicising eclecticism, in which strong influence of medieval revivalism can be noted with neo-Renaissance maintained in the *Rundbogenstil* manner (Fig. 16, 17).

The above mentioned medieval-Gothic reminiscences were not particularly popular among creators of Scandinavian architecture, in striking contrast to neo-Renaissance which remained under the predominant influence of German building. The Nordic neo-Renaissance mainly took shape of various forms of *Rundbogenstil* and neo-stylistic versions of neo-Renaissance, or rather Dutch or Flemish mannerism, in which the influence of Venetian architecture could be discerned. Among excellent examples of the north-Italian and north-European, neo-Renaissance mannerism one must count the *Gesamtkunstwerk*, executed by one of the most remarkable masters of the nineteenth-century historicism and classicism i.e. Christian Hansen the younger, brother to Theophil Hansen. Thus one cannot ignore in the panorama of Copenhagen architecture the building of the Zoological Museum, erected in the years 1863-1869 in the *Rundbogenstil* by Christian Hansen. Even at first glance one can see, that the building, because of its functional similarity, brings to mind an earlier object erected by Theophil Hansen, namely the Viennese Museum of Arms constituting an integral part of the Arsenal built between 1849 and 1856.

The Zoological Museum by Christian Hansen corresponds to the idea of a Renaissance-mannerist *palazzo cortile*, with an essential central courtyard which, in the case of the palace of natural sciences in Copenhagen, was covered with przez tego architekta przy budowie berlińskiego Neues Museum (1843-1855), aż po budynek Akademii Nauk w Budapeszcie (1862-1864) (Ryc. 19).

Sztokholmskie Muzeum F.A. Stülera dostarcza, wcale niebanalnego, spostrzeżenia na temat powszechnego niemal stosowania w architekturze wnętrz obiektów publicznych w XIX wieku, w takich jak np. muzea, ratusze, uniwersytety, teatry itd. – reprezentacyjnych pomieszczeń, których tektonika stwarzała konieczność projektowania wewnętrznych dziedzińców i monumentalnych westybuli, holi i reprezentacyjnych klatek schodowych. Między innymi usytuowanie wyniosłych schodów w centralnej części budynku spotkać można właśnie we wspomnianym Muzeum. Z kolei w westybulu Uniwersytetu w Uppsali, który powstał w manierze późnego *Rundbogenstil*, pomiędzy 1878 a 1887 i zaprojektowany został przez Hermanna Theodora Holmgrena, podwójne schody stanowią istotny element wystroju wnętrza. Podobna prawidłowość występuje w neorenesansowym gmachu Narodowej Galerii Sztuki w Kopenhadze, powstałej podług planów Wilhelma Dahlerupa, ukończonej w 1889-1896. Po przekroczeniu progu zwiedzający wkraczał wprost do ogromnego holu, z którego wiodły imponujące schody w typie *escalier des ambassadeurs,* które miały wprowadzić widza w podniosły nastrój, konieczny przy zwiedzaniu świątyni sztuki, niestety ten zasadniczy element został, z wielką szkodą dla zabytku, zburzony w 1966 roku (Ryc. 20).

Zgoła odmienną kategorię neorenesansu reprezentuje zamek Frederiksborg w Hillerød, położony niedaleko Kopenhagi. Ten monumentalny obiekt należy do największych zabytków tego rodzaju w całej Skandynawii. Zamek powstał na początku XVII wieku w stylu niderlandzkiego renesansu, atoli w 1859 roku w wyniku pożaru uległ niemal całkowitemu zniszczeniu. Obecny stan i wygląd zamku Frederiksborg zawdzięczamy Ferdinandowi Meldahlowi, który przeprowadził w latach 1859-1884 gruntowną rekonstrukcję i restaurację zamku. Podczas odbudowy, według opinii M. Bligaard, obowiązywała reguła *Unité-de-style,* czyli inaczej, uważano, że wnętrza powinne korespondować ze stylem zewnętrznym budowli. Owa pryncypialna jedność zewnętrzna i wewnętrzna nie była wszakże do końca konsekwentna, albowiem F. Meldahl wprowadził w zamku liczne elementy neogotyckie (Ryc. 21). Do pewnego stopnia zamek Frederiksborg porównać można z budynkiem ratusza w Hamburgu, pracy zespołu: Johannes Grotjan, Martin Haller, Bernhard Hanssen Wilhelm Hauser, Hugo Stamma, Gustav Zinnow i firmy Lamprecht – zrealizowanego między rokiem 1887 a 1897.

Jak wiadomo pojęcie stylu było w praktyce bardzo elastyczne, co dotyczyło osobliwie wnętrz zamku Frederiksborg, który w 1884 roku zyskał status muzeum, co implikowało eklektyczny wystrój wnętrz i zgromadzonych zbiorów. Inspiracji, jak to było w zwyczaju w dobie historyzmu, dostarczały wzory zaczerpnięte z różnych wnętrz z zabytkowych obiektów Europy – począwszy od weneckich pałaców, przez niemieckie ratusze, do szkockich czy szwedzkich siedzib szlacheckich. W komnatach zamkowych dominowały ciężkie rzeźbione i polichromowane stropy drewniane, w których nierzadko spotkać można żeliwne ornamenty, bądź w niektórych pomieszczeniach umieszczano późnoneorenesansowe bądź wczesnoneobarokowe plafony.

W drugiej połowie XIX wieku w Skandynawii konstytucyjny status monarchów uległ zasadniczym przeobrażeniom, gdzie dla przykładu w Danii zniesiono w 1849 roku formalny już absolutyzm, którego symbolem był niegdyś oczywiście

a glass construction. The courtyard is surrounded with three-storied arcades, the lower are kept in the neo-Renaissance style, while the second and third storey were designed the neo-Byzantine style which Theophil Hansen had previously introduced in the Museum of Arms (Fig. 18).

Let us interrupt the chronology of the 19th-century architecture in Scandinavia for a moment, as one cannot ignore the services rendered by both the above mentioned Danes in the area of revitalizing the antiquity to the newly reborn, modern Greece. Christian and Theophil Hansen decorated the modern centre of Athens by erecting the objects so important for the culture of the capital of the Hellenic Kingdom as: the University (Ch. Hansen, 1839-1850), the Academy of Sciences (Th. Hansen, 1859-1887) and the National Library (Th. Hansen, 1885-1921). Their austere style of 'Greek Revival' was characterised by almost crystal-clear purity of detail. Experience gained in Greece allowed for developing talents full of invention and imagination which enriched the artistic palette of architecture of both neo-classicism, as eclectic historicism.

In the circle of neo-Renaissance *Rundbogenstil*, taking into account the museum character of the building, there remains the National Museum in Stockholm which was designed in the years 1844-1866 by Friedrich August Stüler, a disciple of K. von Schinkel. The building of the Stockholm museum, which belonged to one of the first of this kind state institutions of public utility, bears clear traces of widely understood central-European school of neo-Renaissance, stretching in its influence from Scandinavia to the Balkans. Animators of that current commonly known as *Rundbogenstil,* which has remained a neo-Renaissance amalgam, were mainly F.A. Stüler, Friedrich von Gärtner, Heinrich Hübsch, Ludwig Persius i Friedrich Bürklein. The building of the National Museum in Stockholm expresses something in the shape of an indirect *genre* of F.A. Stüler, applied by that architect from the construction of the Neues Museum (1843-1855) in Berlin, to the building of the Academy of Sciences in Budapest (1862-1864) (Fig. 19).

The Stockholm Museum of F.A. Stüler provides an original observation on the issue of the quite common use of formal rooms in the interior architecture of public utility objects in the 19th century, such as e.g. museums, town halls, universities, theatres etc. The tectonics of such formal rooms resulted in the necessity to design internal courtyards and monumental vestibules, halls and formal stairwells. For instance, one can encounter a lofty staircase situated in the central part of the building in the already mentioned Museum. And in the vestibule of the University in Uppsala, which was built in the manner of late *Rundbogenstil* between 1878 and 1887, and was designed by Hermann Theodor Holmgren, a double staircase constitute an essential element of interior decoration. A similar regularity occurred in the neo-Renaissance edifice of the National Gallery of Art in Copenhagen, created according to the design of Wilhelm Dahlerup, and completed in 1889-1896. Having crossed the threshold, the visitor entered straight into an enormous hall with imposing staircase in the type of *escalier des ambassadeurs* which was to put the viewer in a sublime mood necessary while visiting a temple of art; unfortunately, this vital element was demolished in 1966, to the detriment of the monument (Fig. 20).

A completely different category of neo-Renaissance is represented by the Frederiksborg castle in Hillerød, located in the vicinity of Copenhagen. That monumental object

Wersal. Dla odmiany na kontynencie europejskim nastąpiła, pod wpływem rewolucji francuskiej i wojen napoleońskich, podstawowa ewolucja, niezależnie od Restauracji, przebiegająca od dawnego *ancien régime*'u do wykształcenia się nowych form rządów autorytarno-paternalistycznych z coraz silniejszymi wpływami parlamentaryzmu, pozostawiając na marginesie pomniejsze satrapie, jak np. Księstwo Parmy i Piacenzy, nie wspominając oczywiście o carskim samodzierżawiu. Druga połowa stulecia zmienia się w sposób zasadniczy w epokę pary, a następnie elektryczności, które zmienią estetyczno-artystyczne oblicze przyszłego społeczeństwa przemysłowego. Nie był to już czas budowy ogromnych rezydencji świadczących o potędze dynastycznej, lecz tworzenia 'pałaców' wczesnej i dojrzałej industrializacji. Zamiast nowych siedzib królewskich powstawały pomniki nowych czasów, takie jak dworce kolejowe, gmachy parlamentów, sądów, czy w ogóle urzędów publicznych, teatry i opery, muzea, itd., itd. Wszelako europejscy dynaści nie zaniechali zupełnie budowy nowych zamków i pałaców, choć były one zjawiskiem raczej wyjątkowym. Przeto fenomenem były, dla przykładu, budzące zachwyt wczoraj i dziś, nowe rezydencje wznoszone przez bawarskiego króla Ludwika II, jak choćby słynny zamek Neuschwanstein koło Füssen, wzniesiony w latach 1868-1892 podług projektu Christiana Janka, Eduarda Rriedla i Juliusa Hofmanna. Neuschwanstein jest neomediewalnym konglomeratem, w którym, z woli mecenasa budowy, autorzy zawarli harmonijnie skomponowane elementy neogotyckie, neoromańskie, starochrześcijańskie, neobizantyńskie i starogermańskie. W podobnym duchu zrealizowane zostały nowe pałace królów Portugalii, jak np. pałac w Sintra wzniesiony podług koncepcji barona Wilhelma Ludwiga von Schwege, który powstał między rokiem 1842 a 1854 oraz pałac królewski w Buçaco koło Aveiro, dzieło architekta hr. Luigi Maniniego, zrealizowane w 1888-1907. Obydwa te pałace są bardzo interesującą i oryginalną formą historycznego eklektyzmu opartą na wzajemnych wpływach hiszpańskiego gotyku, stylu *mudéjar* oraz późnego gotyku i wczesnorenesansowego stylu manuelińskiego. Z powyższych względów, mając na uwadze geopolityczne i historyczno-kulturowe uwarunkowania, warto przytoczyć, niejako *à rebours*, przykład niesłychanie oszczędnego i lapidarnego – eklektycznego pseudomediewalizmu z akcentami neorenesansowymi, jaki stanowi pałacyk Gamlehaugen w Bergen, wystawiony w latach 1898-1900 przez architekta Jensa Zetlitza Monrada Kiellanda. Pałacyk ten zaprojektowany dla norweskiej rodziny królewskiej, odkupiony został przez magnata żeglugowego Christiana Michelsena, co stanowiło swoiste *signum temporis* (Ryc 22). I jeżeli wziąć pod uwagę neoromantyczną atmosferę, w której powstał zamek Neuschwanstein, to dla kontrastu stwierdzić można, że w pałacyku Gamlehaugen widać znaczące wpływy aury pozytywistyczno-symbolicznego końca epoki historyzmu. Z tego względu nie można pominąć zamku cesarskiego w Poznaniu, wystawionego z woli Wilhelma II przez Franza Schwechtena między 1903 a 1913 – jednego z ostatnich przejawów historyzmu. Otóż zamek ten zaprojektowany w stylu neoromańskim posiadał szereg apartamentów, w których dominowały niektóre wnętrza urządzone w stylu melanżu maniery starogermańskiej i staronordyckiej. Do upodobań artystyczno-estetycznych ostatniego już cesarza Niemiec, w kwestii współczesnej mu recepcji stylu staronordyckiego, wypadnie jeszcze powrócić.

I podobnie jak w przypadku budowli o charakterze neośredniowiecznym, również w krajach nordyckich nie wznoszono

is among the largest monuments of this kind in the whole Scandinavia. The castle was built at the beginning of the 17th century in the style of Dutch Renaissance, however, in 1859 it was almost entirely destroyed by fire. The present state and appearance of the Frederiksborg castle we owe to Ferdinand Meldahl who, in the years 1859-1884, carried out a complete reconstruction and restoration of the castle. According to M. Bligaard, during reconstruction the ruling principle was *Unité-de-style,* or in other words, it was believed that interiors should correspond to the style on the outside of the building. However, that principal external and internal unity was not entirely consistent, since F. Meldahl introduced numerous neo-Gothic elements in the castle (Fig. 21). To a certain extent Frederiksborg castle can be compared with the town hall building in Hamburg, the work of a team: Johannes Grotjan, Martin Haller, Bernhard Hanssen, Wilhelm Hauser, Hugo Stamma, Gustav Zinnow and the Lamprecht company – realised between 1887 and 1897.

It is generally known that the notion of style was in reality fairly elastic, which especially referred to the interiors of the Frederiksborg castle that, in 1884, received the status of a museum, which implied eclectic character of interior décor and accumulated exhibits. As was customary during the historicist period, inspiration was found in patterns borrowed from various interiors in historic objects all over Europe – starting from Venetian palaces, through German town halls to Scottish or Swedish nobility manors. In the castle chambers predominant were heavy polychrome timber ceilings, in which one could frequently encounter cast-iron ornaments, or in some rooms one could find late-Renaissance or early-Baroque plafonds.

In the second half of the 19th century, the constitutional status of monarchs in Scandinavia was radically altered, as e.g. the already merely formal absolutism which once was naturally symbolized by Versailles was abolished in Denmark in 1849. On the other hand, under the influence of the French Revolution and Napoleonic Wars the basic evolution took place on the continent, regardless of the Restoration, proceeding from the former *ancien régime* to the establishment of new forms of authoritarian-paternalist rule with increasing influence of parliamentarism, which left smaller satrapies, such as e.g. the Duchy of Parma and Piacenza on the margin, naturally not mentioning the tsarist autocracy. The second half of the century changed dramatically into the epoch of steam, and then electricity, which would trans form the aesthetic-artistic image of the future industrial society. It was no longer time for building enormous residences reflecting dynastic power, but the time for creating 'palaces' of the early and mature industrialisation. Instead of new royal seats, monuments to new times were erected such as: railway stations, parliament or court buildings, or generally of public utility, theatres and opera houses, museums, etc., etc. Nevertheless, European dynasts did not entirely abandon building new castles and palaces, though they became a fairly unique phenomenon. Such a phenomenon were, for instance, new residences built by the Bavarian king Ludwig II, which still arouse admiration, as e.g. the famous Neuschwanstein castle near Füssen, erected in the years 1868-1892 according to the project by Christian Jank, Eduard Rriedl and Julius Hofmann. Neuschwanstein is a neo-medieval conglomerate in which, by the will of the building patron, authors included harmoniously composed neo-Gothic, neo-Romanesque, Early Christian, neo-Byzantine and old-Germanic elements.

nowych rezydencji barokowych, jak to miało miejsce choćby w fundacjach Ludwika II, jak np. neobarokowego pałacu Herrenchiemseeschloss, arch., arch. – Georg Dollmann i Julius Hofmann, 1878-1886 oraz neorokokowego pałacu Linderhof koło Oberammergau, arch. G. Dollmann, 1869-1874. O takich fantazjach architektonicznych królowie Danii, Szwecji i później Norwegii – nie mieli co marzyć. Królewskie rody panujące w Kopenhadze, Sztokholmie i Oslo nie wznosiły już nowych rezydencji, jednak nadal nadawały ton w kwestii wystroju wnętrz. W owym czasie najpopularniejszym tamże był styl II cesarstwa, zwany również stylem Napoleona III, którego animatorami byli Hector-Martin Lefuel, Louis Visconti i Jean-Louis Charles Garnier.

Dobrym przykładem tego rodzaju historyzmu mogą być nowe wnętrza starego pałacu królewskiego w Sztokholmie. W latach 1847-1850 Axel Nyström zaprojektował prywatne apartamenty szwedzkiego następcy tronu, późniejszego króla Karola XV, który interesował się sztuką, aktywnie uczestnicząc w urządzaniu wnętrz mających mieć charakter bardziej *en vogue*. Jego apartamenty składały się z pomieszczeń takich jak pokój chiński, sala herbowa, sypialnie w stylu pompejańskim, buduar w stylu neorokoka, gabinet w siedemnastowiecznej wersji renesansu, palarnia w stylu tureckim i pokój bilardowy w stylu neogotyckim. Niemały wpływ na te wnętrza miał drugi *empire* i poprzedzający go wcześniej styl Ludwika Filipa I Orleańskiego, który wraz ze stale mającym uznaniem biedermeierem cieszył się wzięciem nie tylko u arystokracji, ale i u bogatej burżuazji i plutokracji. Pluralizm form przeniknął również do wystroju sal reprezentacyjnych. Zainspirowana Ch. Percierem i P.F. Fontainem sala balowa „Morze Białe" (Vita Havet) urządzona przez A. Nyströma, została uzupełniona w 1850 roku jadalnią, którą architekt Frederik Wilhelm Scholander urządził w neorenesansowym stylu „gotycko-nordyckim", ze ścianami wyłożonymi dębową boazerią i złoconym kurdybanem z motywami zaczerpniętymi z sag. W latach 1864-1865 F.W. Scholander zaprojektował – utrzymaną w duchu II cesarstwa, w którym neorokoko zyskiwało na powrót na popularności w kręgach dworskich oraz wielkiej i średniej burżuazji – Salę Wiktorii (Ryc. 23).

Frederik Wilhelm Scholander był architektem okresu historyzmu w całym tego słowa znaczeniu. Opanował nieomal wszystkie pseudostyle architektoniczne, w których ulubionym przezeń gatunkiem była kompromisowa maniera neorenesansowo-neobarokowa. W sztokholmskim pałacu królewskim stworzył kolejną paradną klatkę schodową o pięciu podestach, którą ukończono w 1862 roku. Również F.W. Scholander zaprojektował nowy wystrój wnętrz, pochodzącego z XVII wieku, pałacu Ulriksdal koło Solna, na północ od Sztokholmu. Pałac ten nabył w 1856 roku wspomniany już szwedzki *Kronprinz* lokując w nim znaną kolekcję antyków i dzieł sztuki, pochodzących z kraju i z zagranicy. Obiekt ten po przebudowie uzyskał ostateczną formę pozostającą w kostiumie szalenie oszczędnego neobaroku i neorenesansu.

Atoli perłą skandynawskiego stylu francuskiego II cesarstwa pozostaje monumentalny gmach Parlamentu szwedzkiego w Sztokholmie, dzieło Arona Johanssona, zaprojektowane między rokiem 1897 a 1905. Obiekt ten koresponduje, pod względem formalno-stylistycznym, z takimi budynkami publicznymi, jak np. Politechnika w Akwizgranie (Aachen), Robert Cremer, 1865-1870 i Franz Everbeck, 1875-1879 oraz Pałac Sprawiedliwości w Brukseli, Joseph Polaert, 1861-1883. Dekoracje wnętrz Parlamentu reprezentuje typowy dla tego okresu eklektyzm, uzupełniony przez A. Johanssona, charak-

New palaces of the kings of Portugal were realised in a similar spirit, such as e.g.: the palace in Sintra built according to the concept of baron Wilhelm Ludwig von Schweg, which was erected between 1842 and 1854, and the royal palace in Buçaco near Aveiro, a work by architect count Luigi Manini, realised in 1888-1907. Both those palaces are very interesting and original forms of historic eclecticism based on mutual influences of Spanish Gothic, the *mudéjar* style, late Gothic and the early-Renaissance Manueline style. Because of the above, and considering geopolitical, historical and cultural conditions, it is worth quoting almost *à rebours* the example of extremely economical and lapidary – eclectic pseudo-medievalism with neo-Renaissance accents embodied by the Gamlehaugen palace in Bergen, erected in the years 1898-1900 by architect Jens Zetlitz Monrad Kielland. The palace, designed for the Norwegian royal family, was purchased by a shipping tycoon Christian Michelsen, which was indeed a *signum temporis* (Fig. 22). And when one considers the neo-romantic ambience in which the Neuschwanstein castle was built then, in contrast, one could state that the Gamlehaugen palace shows visible influence of the aura of the positivist-symbolic end of the historicism epoch. For that reason it would be unthinkable to ignore the imperial castle in Poznan, built by the wish of Wilhelm II, by Franz Schwechten between 1903 and 1913 – one of the last manifestations of historicism. The castle, designed in the neo-Romanesque style, possessed several apartments in which some interiors furnished in a stylistic mélange of Old-Germanic and Old-Norse manner were predominant. We will return again to the issue of artistic and esthetic preferences of the last German Emperor concerning the reception of the Old-Norse style in his times.

Similarly as in the case of buildings of neo-medieval character, no new neo-Baroque residences were erected in the Nordic countries, as was the case e.g. in the foundations of Ludwig II, such as: the neo-Baroque Herrenchiemseeschloss, designed by Georg Dollmann and Julius Hofmann, 1878-1886; and the neo-rococo Linderhof palace near Oberammergau, designed by G. Dollmann, 1869-1874. Such architectonic fantasies Kings of Denmark, Sweden and later Norway could but dream about. Royal families ruling in Copenhagen, Stockholm and Oslo no longer had new residences erected, though they still set the tone of interior decoration. At that time the most popular there was the style of II Empire, also known as the style of Napoleon III, whose animators were: Hector-Martin Lefuel, Louis Visconti and Jean-Louis Charles Garnier.

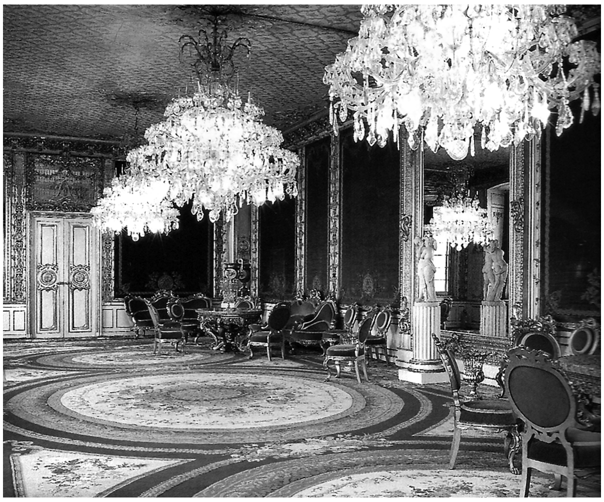
A good example of that kind of historicism can be the new interiors of the old royal palace in Stockholm. In the years 1847-1850, Axel Nyström designed private apartments of the heir to the throne of Sweden, later King Charles XV, who was interested in art and actively participated in furnishing the interiors which were to be more *en vogue*. His apartments consisted of such rooms as: a Chinese room, a coat-of-arms room, bedrooms in the Pompeian style, a boudoir in the neo-rococo style, a study in the seventeenth-century version of the Renaissance, a smoking room in Turkish style and a billiards room in the neo-Gothic. The style of the II *Empire* and the preceding it style of Louis Philippe of Orleans which, together with the constantly admired Biedermeier, was popular not only among the aristocracy, but also among the affluent bourgeoisie and plutocracy had a great impact on the interiors. Pluralism of form penetrated also to the interior decor of formal rooms.

Ryc. 19. Muzeum Narodowe. Sztokholm. F.A. Stüler, 1844-1866
Fig.19. National Museum. Stockholm. F.A. Stüler, 1844-1866

Ryc. 20. Narodowa Galeria Sztuki. Kopenhaga. W. Dahlerup, 1889-1896
Fig.20. National Gallery of Art. Copenhagen. W. Dahlerup, 1889-1896

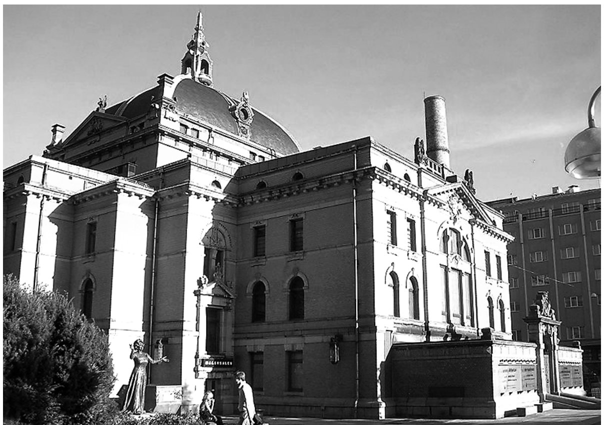
Ryc. 21. Zamek Frederiksborg. Hillerød. F. Meldahl, 1859-1884
Fig.21. Frederiksborg Castle. Hillerød. F. Meldahl, 1859-1884

Ryc. 22. Pałacyk Gamlehaugen. Bergen. J.Z.M. Kielland, 1898-1900
Fig.22. Gamlehaugen Palace. Bergen. J.Z.M. Kielland, 1898-1900



Ryc. 23. Pałac Królewski. Sala Wiktorii. Sztokholm. F.W. Scholander, 1864-1865
Fig.23. Royal Palace. Victoria's Room. Stockholm. F.W. Scholander, 1864-1865

Ryc. 24. Parlament. Oslo. A. Johansson, 1897-1905
Fig.24. Parliament. Oslo. A. Johansson, 1897-1905



Ryc. 25. Teatr Narodowy. Oslo. H. Bulla, 1899
Fig.25. National Theatre. Oslo. H. Bulla, 1899

Ryc. 26. Królewski Teatr Dramatyczny. Sztokholm. F. Lilljekvist, 1908
Fig.26. Royal Drama Theatre. Stockholm. F. Lilljekvist, 1908

terystycznymi elementami i motywami zaczerpniętymi ze swojsko-regionalnej kultury baśniowo-ludowej, osobliwie co się tyczy Norwegii, która pozostawała jeszcze w unii ze Szwecją. Do problemów rozwijającego się w Skandynawii rozkwitu architektury opartej na wzorach narodowo-wernakularnych przyjdzie pora powrócić jeszcze na chwilę. Tymczasem z punktu widzenia chronologii dziejów sztuki nordyckiej należy wspomnieć dodatkowo o dwóch obiektach. Pierwszym, uznanym za neorenesansowo-neobarokowy, jest Teatr Narodowy w Oslo, usytuowany pomiędzy pałacem królewskim, Parlamentem i Uniwersytetem. Gmach Teatru Narodowego w Oslo zaprojektował w 1899 roku Henrik Bull. Drugim z kolei jest budynek Królewskiego Teatru Dramatycznego w Sztokholmie, którego autorem jest Frederik Lilljekvist. Obiekt ten, który zrealizowany został w 1908 roku, ma również cechy neobarokowego historyzmu z domieszką secesji, czy lepiej – Art Nouveau, który porównawczo można zestawić, choćby ze względu na kulturowe powinowactwo, z architekturą Rygi, jak np.: Łotewski Teatr Narodowy, August Reinberg, 1900-1902 i Państwowe Muzeum Sztuki, Wilhelm Neumann, 1903-1905 (Ryc. 24, 25, 26).

W panoramie architektury nordyckiej nie można zapomnieć o, posiadającym wielowiekowe tradycje, dziedzictwie budownictwa drewnianego. Tę niesłychanie oryginalną spuściznę przypomina, znajdujący się obecnie w Polsce, kościół ewangelicki w Karpaczu, położony u stóp Śnieżki, czyli tak zwana powszechnie świątynia Wang (Vang). To sanktuarium, powstałe na przełomie XII i XIII wieku w południowej Norwegii, przypomina o mnogości obiektów tego typu na tym obszarze, z których ostało się do czasów obecnych ledwie kilkaset, tym bardziej zatem kościół w Karpaczu jest wartościową i cenną pamiątką norweskiej kultury narodowej. Przeniesienie tego autentycznego zabytku cywilizacji Celtów i Wikingów należy zawdzięczać artystycznemu i historycznemu mecenatowi pruskiego króla Fryderyka Wilhelma IV, za którego sprawą kościół Wang znalazł się wprzódy w Szczecinie, następnie w Berlinie, aby ostatecznie zostać ulokowany w Karpaczu w 1844 roku. Wang należy do kategorii świątyń klepkowo-słupowych (niem. – *Stabkirche,* nor. – *stavkirke*), rozpowszechnionych na północy Niemiec i w Skandynawii, z tym tylko wyjątkiem, że te ostatnie, szczególnie w odniesieniu do wystroju ogólnego i wnętrz oraz dekoracji i ornamentu, nawiązują do staronorweskiej sztuki, opartej na pradawnych wzorach pozostających na granicy mitu i legendy. Te ostatnie fenomeny staną się integralną częścią dziewiętnastowiecznego odrodzenia narodowo-ludowego wernakularyzmu w architekturze krajów północnej Europy, gdzie wiodącą rolę odegrała drewniana snycerka, zyskując tamże na ogromnej popularności i znaczeniu, szczególnie w drugiej połowie XIX wieku.

Wystawy światowe, modne zwłaszcza po około 1850 roku, obudziły wśród biorących w nich udział państw potrzebę prezentacji narodowych tradycji. Znalazło to również wyraz w kształcie pawilonów wystawowych, po raz pierwszy na Światowej Wystawie Paryskiej w 1867 roku, kiedy to nie tylko Rosja, ale także Szwecja i Norwegia wzbudzały zainteresowanie pawilonami z drewna. Pawilon szwedzki był pomniejszoną kopią historycznego drewnianego budynku z XVII wieku, częściowo krytego gontem. Dla odmiany pawilon norweski miał kształt *stabbur* – jednego z typowych zabudowań w gospodarstwie chłopskim (rodzaj magazynu), z wysuniętą górną kondygnacją, opartą na słupach tworzących podcienie. Z kolei na Wiedeńskiej Wystawie Światowej w 1873 roku Szwecja ponownie

Inspired by Ch. Percier and P.F. Fontain, the ballroom "White Sea" (Vita Havet) furnished by A. Nyström, was supplemented in 1850 by a dining room which architect Frederik Wilhelm Scholander organised in the neo-Renaissance "Gothic-Nordic" style, with walls laid in oak panelling and gilded cordovan with motifs borrowed from sagas. In the years 1864-1865, F.W. Scholander designed the Victoria Room maintained in the spirit of the II Empire in which neo-rococo again gained in popularity among the court circles and among the upper and middle bourgeoisie (Fig. 23).

Frederik Wilhelm Scholander was an architect of the historicism period in the full sense of the word. He mastered almost all architectural pseudo-styles among which his favourite genre was a compromise neo-Renaissance – neo-Baroque manner. In the royal palace in Stockholm, he created yet another formal staircase with five landings which was completed in 1862. F.W. Scholander also designed new interior decoration for the 17th-century Ulriksdal palace near Solna, to the north of Stockholm. In 1856 the palace was purchased by the already mentioned Swedish *Kronprinz* and located there his well-known collection of antiquities and masterpieces of native and foreign art. After alterations the object acquired its final form, remaining in the costume of extremely frugal neo-Baroque and neo-Renaissance.

However, the pearl of Scandinavian style of the French II Empire has been the monumental edifice of the Swedish Parliament in Stockholm, the work of Aron Johansson, designed between 1897 and 1905. In formal-stylistic respect, the object corresponds with such public utility buildings as e.g.: the Polytechnic in Aachen, by Robert Cremer, 1865-1870, and Franz Everbeck, 1875-1879; and the Palace of Justice in Brussels, by Joseph Polaert, 1861-1883. Interior decoration in the Parliament represents eclecticism, typical for that period, supplemented by A. Johansson with characteristic elements and motifs borrowed from the native-regional legendary-folk culture, especially as far as Norway was concerned which still remained in union with Sweden. Problems of the heyday of architecture based on national-vernacular patterns, developing in Scandinavia, will again be addressed later. Now, considering the chronology of the history of Nordic art, one should add some remarks concerning two objects. The first is the National Theatre in Oslo, regarded as neo-Renaissance – neo-Baroque, located between the royal palace, the Parliament and the University. The edifice of the National Theatre in Oslo was designed in 1899 by Henrik Bull. The other is the building of the Royal Drama Theatre in Stockholm, the author of which was Frederik Lilljekvist. The object, realised in 1908, show also features of neo-Baroque historicism with a touch of Secession, or better – Art Nouveau, can be compared, e.g. because of its cultural affinity, with the architecture of Riga, such as e.g.: the Latvian National Theatre by August Reinberg, 1900-1902; and the National Museum of Art by Wilhelm Neumann, 1903-1905 (Fig. 24, 25, 26).

In the panorama of Nordic architecture, one cannot forget the heritage of timber building which can boast centuries-old traditions. That extremely original legacy is alluded to by the evangelical church in Karpacz, Poland, located at the foot of Mt. Śnieżka, commonly known as the Wang Temple. That sanctuary, erected at the turn of the 12th and 13th century in southern Norway, recalls the abundance of objects of that type in the area, of which barely a few hundred have remained till today, thus making the church in Karpacz even more valuable monument of the Norwegian national culture. The transfer of

Ryc. 27. Dworzec kolejowy. Bergen. J.Z.M. Kielland, 1900-1913
Fig.27. Railway Station. Bergen. J.Z.M. Kielland, 1900-1913

Ryc. 28. Ratusz. Kopenhaga. M. Nyrop, 1892-1902
Fig.28. Town Hall. Copenhagen. M. Nyrop, 1892-1902

Ryc. 29. Duńska Królewska Biblioteka. Kopenhaga. H.J. Holm, 1906
Fig.29. Danish Royal Library. Copenhagen. H.J. Holm, 1906

zaprezentowała drewniane budynki, tym razem spełniające nowoczesne funkcje, przystosowane również do projektowania reprezentacyjnych willi (dworów), w których widoczne były owe wysunięte nadwieszenia w górnych partiach obiektów. Architektura ta promowana była jako szwedzko-norweski styl narodowy.

Drewniana willa z wysuniętym dachem, przewiewnymi altanami i jasnymi pokojami nadawała się szczególnie na letnie mieszkanie, dające możliwość kontaktu z naturą. W pewnym

Ryc. 30. Ratusz. Sztokholm. R. Östberg, 1908-1923
Fig.30. Town Hall. Stockholm. R. Östberg, 1908-1923

Ryc. 31. Ratusz, dziedziniec. Sztokholm. R. Östberg, 1908-1923
Fig.31. Town Hall, courtyard. Stockholm. R. Östberg, 1908-1923

this authentic monument of the Celtic and Viking civilisation must be attributed to the artistic and historical patronage of the Prussian king Frederic Wilhelm IV, by whose orders the Wang church was first moved to Szczecin and then to Berlin, to be finally located in Karpacz in 1844. Wang belongs to the category of stave churches (Ger. – *Stabkirche,* Nor. – *stavkirke*), popular in northern Germany and Scandinavia, apart from the fact that the latter, especially concerning their overall and interior decoration as well as ornaments, allude to the Old-Norse art based on ancient motifs verging on the myth and legend. The latter phenomena would become an integral part of the nineteenth-century revival of national-folk vernacularism in architecture of the north-European countries, in which the leading role was played by woodcarving gaining immense popularity and significance there, especially in the second half of the 19th century.

World Fairs, very fashionable after around 1850, aroused the need to present national traditions among the countries participating in them. It was also expressed in the shape of exhibition pavilions, for the first time on the World Fair in Paris in 1867, when not only Russia, but also Sweden and Norway evoked interest with their timber pavilions. The Swedish Pavilion was a smaller copy of a historic timber building from the 17th century, partially covered with wooden shingle. The Norwegian Pavilion, on the other hand, was in the shape of a *stabbur* – a typical building in a peasant farmstead (a kind of storage building), with the upper storey overhanging the lower, supported by posts creating arcades. During the World Fair in Vienna in 1873, Sweden again presented timber buildings, but this time fulfilling modern functions, and also adopted to stopniu znosiła podział na wnętrze i część zewnętrzną, która była wkomponowana w otaczającą ją przyrodę. Rzut poziomy był zazwyczaj asymetryczny, z centralnie położonym salonem lub pokojem „rodzinnym". To duże pomieszczenia porównywano z „salą wikingów", którą Eugène Emmanuel Viollet-le--Duc opisał w swojej książce z 1875 roku *Histoire de l'habitation humaine*, jako typ pokoju charakterystycznego dla Skandynawii. Owo pomieszczenie cechował wysoki sufit, otwarty kominek, nagie, niemalowane drewniane ściany, zdobione rzeźbionymi motywami, czasem malowanymi. Proste materiały stanowiły ważny element wystroju wnętrz, a mieszkańcy bardzo często nosili stroje staronordyckie.

W obrębie tego *genre*'u wyróżniają się dwa, bodaj najbardziej charakterystyczne, przykłady, z których jeden jest potwierdzeniem estetycznego etosu umiaru, drugi zaś – świadectwem drewnianego budownictwa, w którym łączono funkcjonalność z wyszukaną okazałością. Mamy tu na myśli pierwszą ze wspomnianych egzemplifikacji, czyli Willę Bråvalla w Gustavsberg, na wschód od Sztokholmu, zbudowaną w latach 1869-1870 przez J.F. Söderlunda, w której wnętrza zaprojektował Magnus Izeaus. Oryginalny klimat tej i podobnych willi podkreśla całość wyposażenia w stylu staronordyckim, gdzie wzdłuż ścian zamontowano ławy, a ciężkie dębowe meble o nieco kanciastych formach zdobione były motywami smoków i run. Malowany fryz z motywami z sag zdobił ściany poniżej sufitu. Środek salonu zajmował długi masywny stół.

Drugim przykładem filiacji stylu staronordyckiego jest pałacyk myśliwski (*Jagdhof*) w Rominten (pol. – Rominty, obecnie ros. – Krasnolesie w obwodzie Kaliningradzkim, w dawnych Prusach Wschodnich). Pałacyk ten powstał na polecenie cesarza Wilhelma II, którego ulubionym rewirem myśliwskim była Puszcza Romnicka. I podobnie jak w przypadku Karpacza, dwór *Kaisera* pierwotnie wykonany został w Norwegii, a następnie złożony z gotowych elementów i ustawiony w 1891 roku w Rominten. Autorem projektu był norweski architekt, działający przez wiele lat w Hanowerze, Holm Hansen Munthe. Obok pałacyku zrealizowano, zaprojektowaną przez tego samego projektanta, kaplicę św. Huberta, nawiązującą do maniery kościołów klepkowych, i ustawioną w 1893 roku. W czasie II wojny światowej obiekty te zostały kompletnie zniszczone, jednakże dzięki zachowanej dokumentacji dużo można powiedzieć o atmosferze, w jakiej powstał ów staronordycki pałacyk. Niektóre wnętrza w Rominten żywo przypominają neo-mediewalny (starogermański) nastrój, jaki możemy odczuć w pomieszczeniach zamkowych Neuschwanstein, jak np. w Sali Bardów lub w Gabinecie Królewskim.

Holm Hansen Munthe wydał wspólnie z Lorentzem Dietrichsonem opublikowaną w Berlinie w 1893 roku pracę pt. *Die Holzbaukunst Norwegens in Vergangenheit und Gegenwart*, na temat norweskiej sztuki rzeźbienia w drewnie i architektury kościołów klepkowych. To monumentalne dla odrodzenia kultury norweskiej dzieło uważane jest za podstawę stworzonego przez, między innymi, H.H. Munthe'a tzw. „stylu smoczego" *(Drachenstil)* w norweskim budownictwie drewnianym, który preferował również Wilhelm II. Wspomniana książka była wówczas dowodem na to, że norweska architektura wkroczyła do Niemiec – nareszcie inspiracje płynęły w odwrotnym kierunku.

Preponderancja kultury niemieckiej, szczególnie co się tyczy jej północnych regionów, uwzględniając stosowne *toutes proportions gardées* wraz z wyjątkowo znaczącym, etniczno-kulturowym powinowactwem z narodami nordyckimi, trwającym

designing formal villas (manors), in which those protruding overhanging upper storeys were also visible. That architecture was advertised as the Swedish-Norwegian national style.

A timber villa with an overhanging roof, airy gazebos and bright rooms was particularly suitable for summer accommodation, as it offered the opportunity of close contact with nature. To a certain degree, it cancelled the division into the interior and the exterior part which was incorporated into the surrounding nature. The plan of such building was usually asymmetrical, with a centrally located salon or a family living-room. That huge room was compared to a Viking hall" described by Eugène Emmanuel Viollet-le-Duc in his book from 1875, entitled *Histoire de l'habitation humaine*, as a type of room characteristic for Scandinavia. That room was characterised by a high ceiling, open fireplace, bare, unpainted timber walls decorated with carved motifs which were sometimes painted. Simple materials constituted an important element in interior decoration, ad inhabitants frequently wore Old-Norse clothes.

Within that *genre* two perhaps most characteristic examples stand out, one of which is a confirmation of the aesthetic ethos of moderation, while the other bears witness to timber constructions in which functionality was combined with refined grandeur. The first example we have in mind is the Villa Bråvalla in Gustavsberg, east of Stockholm, built in the years 1869-1870 by J.F. Söderlund, while interiors were designed by Magnus Izeaus. The original atmosphere of this and similar other Villas is emphasised by its furnishings in Old-Norse style, where benches were fitted along the walls, and heavy oak wood furniture with slightly angular forms were decorated with motifs of dragons and runes. A painted frieze with motifs from sagas decorated the walls below the ceiling. The centre of the salon was occupied by a long massive table.

The other example of filiation in the old-Nordic style is a hunting lodge (*Jagdhof*) in Rominten (Pol. – Rominty, currently Rus. – Krasnolesie in the Kaliningrad district, in the former East Prussia). The palace was commissioned Emperor Wilhelm II, whose favourite hunting grounds was the Romincka Forest. And like in the case of Karpacz, *Kaiser's* Manor was originally made in Norway, and later assembler from ready-made elements and set up in Rominten in 1891. The author of the project was Holm Hansen Munthe, a Norwegian architect who worked in Hannover for many years. The Chapel of St. Hubert designed by the same architect, and alluding to the manner of stave churches, was realised near the palace and set up in 1893. During World War II the objects were completely destroyed, but owing to the preserved documentation much can be said about the atmosphere in which that old-Nordic palace was created. Some interiors in Rominten vividly recalled the neo-medieval (old-Germanic) ambience which we can sense in the castle chambers of Neuschwanstein, such as e.g.: in the Bard Chamber or the Royal Cabinet.

Holm Hansen Munthe together with Lorentz Dietrichson were the authors of the work published in Berlin in 1893, entitled *Die Holzbaukunst Norwegens in Vergangenheit und Gegenwart*, concerning the Norwegian art of woodcarving and stave church architecture. That monumental work contributed to the revival of Norwegian culture and is regarded as the basis of the so called "dragon style" *(Drachenstil)* in Norwegian timber construction, created by H. H. Munthe and others, and favoured by Wilhelm II. The above mentioned book was then a proof that Norwegian architecture came into Germany – so finally inspirations flower in the other direction.

do czasów mniej więcej po II wojnie światowej, wiodły do jednoczesnej emancypacji i równoczesnego rozkwitu cywilizacyjno-kulturowego krajów północy. Europejski sukces wybitnych Nordyków, którzy w ostatniej ćwierci XIX wieku włożyli trwały wkład do ogólnoświatowego dziedzictwa, towarzyszył szczególny *esprit*, a jego twórcami byli między innymi Edvard Hagerup Grieg, Jean Sibelius, Henrik Ibsen czy August Strinberg. Twórczość tych znakomitych muzyków, literatów, poetów, plastyków i architektów zakorzeniona jest po części w pradawnej kulturze staronordyckiej, która generowała kolejne prądy artystyczno-estetyczne, takie jak narodowy romantyzm, symbolizm, naturalizm oraz wczesny ekspresjonizm. Ta intrygująca i pełna wewnętrznego napięcia sztuka przyniosła na przełomie XIX i XX wieku szczególnie dorodne owoce, spajając tradycję z nadciągającym modernizmem.

Odrębną kwestią kultury nordyckiej, stanowiącą w istocie wielką narrację historyczną na przełomie stuleci, jest architektura Finlandii, znamionująca przebudzenie narodowe mające ambicję stworzenie dystansu do imperialnej Rosji. Innymi słowy, chodziło tu głównie o utrzymanie autonomii kraju wraz z etniczno-kulturowym odrodzeniem. W ostatecznym rezultacie w fińskiej architekturze stopił się jak tyglu nowy modus, w którym powstała eklektyczno--innowacyjna pospołu nowa forma zawierająca elementy ludowe i staronordyckie, wplecione w neośredniowieczne motywy. W sporej mierze owa synteza zaczerpnięta została z neoromańskiej szkoły wykreowanej, między innymi, przez Henry'ego Hobsona Richardsona bądź Franza Schwechtena. Wyrazem takiego regionalizmu, który promieniował również na budownictwo w Rydze, jest działalność np. Larsa Soncka – gmach Towarzystwa Telefonicznego, (1905); Bank Narodowy, (1903-1904); Hermana Geselliusa, Armasa Lindgrena i Eliela Saarinena – Muzeum Narodowe, (1905-1910) – wszystkie wystawione w Helsinkach.

Przeto podążając tym tropem warto zakończyć niniejszy *tour d'horizon* kilkoma egzemplifikacjami uświęconego obyczajem historyzmu, a zarazem innowacyjnego eklektyzmu. Przegląd takich tendencji w architekturze nordyckiej leżących na pograniczu tych stuleci rozpocząć wypada od pracy, wykształconego w Berlinie, Jensa Zetlitza Monrada Kiellanda, który wzniósł w Bergen w latach 1900-1913 gmach dworca kolejowego (Ryc. 27). Budynek ten nosi znamiona secesji berlińskiej, jednakże obiekt ów osadzony jest w tradycji budownictwa staronordyckiego zaadaptowanego do estetycznych wymogów przełomu stuleci. Świadczą o niej wymownie liczne elementy nadające szczególne piętno w architekturze całego nieomal obszaru estetyczno-kulturowego, nie wyłączając Finlandii oraz obecnych Estonii i Łotwy. Widzimy zatem w fasadzie dworca, obok niezliczonych rozwiązań tego typu, przykładowo otwory okienne zamknięte łukiem geometryczno-parabolicznym o schemacie wydłużonego pentagonu. Okna i wnętrza o takiej formie stosowane są nadal niemal na całym interesującym nas tutaj terytorium. Tego rodzaju konstrukcje wnętrz i kompozycje elewacji znane są szeroko za sprawą, szczególnie w wersji fińskiego, estońskiego oraz łotewskiego – narodowego romantyzmu, w którym istotną rolę odgrywała tradycja sztuki mediewalno-regionalnej i ludowej zarazem. Szeroko znanym w literaturze przedmiotu, wzorcowym przykładem takiego *genre*'u jest aranżacja wnętrz wilii w fińskiej miejscowości Kirkkonummi, zbudowanej przez Hermana Geselliusa, Armasa Lindgrena i Eliela Saarinena w 1902 roku. Następnym uzewnętrznionym przejawem narodowego romantyzmu, tym

Preponderance of the German culture, especially concerning its northern regions, and taking into account suitable *toutes proportions gardées* with the extremely significant ethnic-cultural affinity with the Nordic nations lasting till the times after World War II, led to simultaneous emancipation and civilisational-cultural flourishing of the northern countries. European success of eminent Nordic artists who, during the last quarter of the 19th century, made a permanent contribution to the world heritage, was accompanied by a specific *esprit* whose creators were, among others: Edvard Hagerup Grieg, Jean Sibelius, Henrik Ibsen, or August Strinberg. The works of those wonderful musicians, writers, poets, artists and architects are partially rooted in the ancient Old-Norse culture which generated subsequent artistic-aesthetic currents, such as: national romanticism, symbolism, naturalism and early expressionism. That art, intriguing and full of internal tension, bore particularly fine-looking fruits at the turn of the 19th and 20th century, by merging tradition with the imminent modernism.

A separate issue in the Nordic culture, actually constituting a great historic narrative at the turn of the centuries, is constituted by the architecture of Finland signifying a national awakening which had an ambition of creating a distance towards Imperial Russia. In other words, the main idea was maintaining the autonomy of the country during its ethnic-cultural revival. The ultimate result was that Finnish architecture served as a melting-pot for merging the new modus, in which a new both eclectic-innovative forma was created that included folk and Old-Norse elements interwoven with neo-medieval motifs. To a great extent that synthesis was borrowed from the neo-Romanesque school created, among others, by Henry Hobson Richardson or Franz Schwechten. Such regionalism which also radiated onto building in Riga was expressed in the works of e.g.: Lars Sonck – the edifice of the Telephonic Society, (1905); the National Bank, (1903-1904); Herman Gesellius, Armas Lindgren and Eliel Saarinen – the National Museum, (1905-1910) – all of them erected in Helsinki.

Following this trail, it might be worthwhile to end this *tour d'horizon* with a few examples of historicism sanctified by tradition and, at the same time, of innovative eclecticism. A review of such tendencies in the Nordic architecture of the turn of the centuries should undoubtedly commence with the work of Jens Zetlitz Monrad Kielland, educated in Berlin, who erected the railway station in Bergen in the years 1900-1913 (Fig. 27). The building show signs of the Berlin Secession, nevertheless, the object is rooted in the tradition of Old-Norse constructions adapted to the aesthetic requirements of the turn of the 19th and 20th century. The evidence of it can be perceived in the numerous elements which imprinted the specific mark of the epoch on the architecture of almost the whole aesthetic-cultural area, including Finland and the present-day: Estonia and Latvia. Thus, besides countless solutions of this type, in the facade of the railway station we can see window openings enclosed with geometric-parabolic arch in the shape of an elongated pentagon. Windows and interiors in that form have still been in use in almost the whole territory of our interest. Such a type of interior construction and composition of elevations is widely known because of the national romanticism, especially in its Finnish, Estonian and Latvian version – in which a significant role was played by the tradition of medieval-regional and, at the same time, folk art. Widely known in the literature of the subject, a model example of such *genre* is the interior arrangement in a villa built by Herman Gesellius, Armas Lindgren razem zaprojektowanym w duchu antycypującym modernizm, jest dworzec kolejowy w Helsinkach. Obiekt ten wybudowany został między rokiem 1904 a 1919 przez Eliela Saarinena, który wkrótce, po rozstrzygnięciu konkursu, zyskał międzynarodową reputację. Helsiński dworzec zaprojektowany został niejako z jednej strony w narodowo-późnoromantycznej treści, z drugiej zaś powstał w formie wczesnoracjonalistycznego monumentalizmu, w którym znać filiacje schyłkowej Art Nouveau oraz wczesnej Art Déco.

Z kolei w Kopenhadze powstały na przełomie XIX i XX wieku, doniosłe dla duńskiej architektury, przykłady dojrzałego eklektyzmu, w których odczytać można oddziaływanie schyłkowego historyzmu w twórczości artystów holenderskich, którzy mimo przenikania u nich fascynacją modernizmem, nadal byli wierni tradycji. Pierwszy z nich to, będący chlubą duńskiej architektury, kopenhaski ratusz – dzieło Martina Nyropa wybudowane w latach 1892-1902 (według M. Bligaard oraz Williama J.R. Curtisa), zaprojektowany został zgodnie z ideą narodowego romantyzmu i oparty był, między innymi, na budowlach w takich miejscach, jak Siena, Rawenna, Werona, Marienburg, Lund. Kopenhaski ratusz jest najbliższy współczesnym wzorom odnoszącym się do neomediewalnego neorenesansu, któremu hołdował przykładowo Hendrik Petrus Berlage, autor Giełdy w Amsterdamie, pochodzącej z 1898-1903 lub też Henry Hobson Richardson, czy twórca niezrealizowanego niestety projektu Pałacu Pokoju z 1905 roku, Willem Kromhout (Ryc. 28).

Drugim z tych obiektów jest gmach Duńskiej Królewskiej Biblioteki w Kopenhadze, autorstwa innego pioniera nieawangardowej nowoczesności zespolonego z zachowawczym narodowym romantyzmem, Hansa Jørgena Holma. Budowla ta ukończona została w 1906 roku i pozostaje wyraźnie pod wpływem holenderskich architektów z kręgu Petrusa Josephusa Hubertusa Cuypersa, którego może być bardzo wymowne porównanie z takimi amsterdamskimi obiektami, jak Muzeum Królewskie (1876-1885) oraz Dworzec Kolejowy, wspólna praca P.J.H. Cuypersa, Adolfa Leonarda van Glendta i Leonarda Johannusa Eijmera (1882-1889). I analogicznie jak w przypadku Duńskiej Królewskiej Biblioteki, również architekturę tych gmachów inspirował narodowy romantyzm, któremu nadawały ton neośredniowieczne i neorenesansowe (głównie mając na myśli manieryzm niderlandzki) reminiscencje historyczne. Niewątpliwą ozdobą tej największej książnicy w Skandynawii jest skomponowana przez H.J. Holma centralna aula, będąca kopią kaplicy pałacowej Karola Wielkiego (Charlemagne) w Akwizgranie, dowodząca o ostatnich już przejawach rewiwalizmu (*nomen omen* – renesansu karolińskiego) konserwatywnego historyzmu (Ryc. 29).

Przekrój wybranych problemów dziewiętnastowiecznej architektury krajów nordyckich wypada zamknąć symbolicznym akordem w postaci imponującego gmachu Ratusza w Sztokholmie. Obiekt ten powstał w latach 1908-1923 podług planów wykonanych przez Ragnara Östberga i przyjmuje się na ogół (np. W.J.R. Curtis i D. Watkin), iż ogólna stylistyka budynku oparta jest w głównej mierze na idei weneckiego Pałacu Dożów. Atoli maniera zastosowana przez R. Östberga zaczerpnięta została przeważnie za sprawą recepcji mitycznej narodowej przeszłości z odniesieniami do szwedzkiego neomediewalizmu i neorenesansu, plasujących się między tradycyjnym historyzmem a umiarkowaną nowoczesnością. Innymi słowy, użyty przez architekta, zakorzeniony w tradycji, eklektyczny słownik – stał się swoistym *intermezzo* między kulminacją

and Eliel Saarinen in 1902 in a Finnish town of Kirkkonummi. The next manifestation of national romanticism, this time designed in the spirit anticipating modernism, is the railway station in Helsinki. The object was built between 1904 and 1919, by Eliel Saarinen who, soon after the competition had been adjudicated, gained international renown. The railway station in Helsinki was designed, on the one hand, with the national-late-romantic content, while on the other, it was created in the form of early-rationalist monumentalism in which filiations of decadent Art Nouveau and early Art Déco can be discerned.

At the turn of the 19th and 20th century in Copenhagen, there appeared examples of mature eclecticism, momentous for Danish architecture, in which one could see the impact of decadent historicism in the works of Dutch artists who, despite being fascinated with modernism, were still faithful to tradition. The first of them is the Town Hall in Copenhagen, the pride of Danish architecture, the work by Martin Nyrop built in the years 1892-1902 (according to M. Bligaard and William J.R. Curtis), designed in accordance to the idea of national romanticism and based on buildings in such places as: Siena, Ravenna, Verona, Marienburg, Lund. The Town Hall in Copenhagen is the closest to the present-day models alluding to neo-medieval neo-Renaissance, which was adhered to by e.g. Hendrik Petrus Berlage, the author of the Stock Exchange in Amsterdam, built in the years 1898-1903, or Henry Hobson Richardson, or the creator of the never realised project of the Peace Palace from 1905, Willem Kromhout (Fig. 28).

Another such object is the edifice of the Danish Royal Library in Copenhagen, the author of which was yet another pioneer of non-avant-garde modernity combined with conservative national romanticism, Hans Jørgen Holm. The construction was completed in 1906, clearly remaining under the influence of Dutch architects from the circle of Petrus Josephus Hubertus Cuypers, and can be compared with such objects in Amsterdam as: the Royal Museum (1876-1885) and the Railway Station, a result of co-operation between P.J.H. Cuypers, Adolf Leonard van Glendt and Leonard Johannus Eijmer (1882-1889). Analogically as in the case of the Danish Royal Library, the architecture of those edifices was also inspired by national romanticism the tone of which was set by neo-medieval and neo-Renaissance (meaning mainly Dutch mannerism) historical reminiscences. The pride of that largest library in Scandinavia is, composed by H.J. Holm, the central hall which is a copy of the palace chapel of Charlemagne in Aachen, thus bearing evidence of the last manifestations of revivalism (*nomen omen* – Carolingian revival) of conservative historicism (Fig. 29).

To finish this selection of problems of the nineteenth-century architecture of the Nordic countries, its cross section ought to be ended on a symbolic strong note in the form of the imposing edifice of the Town Hall in Stockholm. That object was erected in the years 1908-1923 according to the plans made by Ragnar Östberg, and it is generally accepted (e.g. W.J.R. Curtis and D. Watkin), that the overall stylistics of the building is mostly based on the concept of the Venetian Doge's Palace. However, the manner applied by R. Östberg, was mainly conveyed by means of reception of the mythical national past with allusions to Swedish neo-medievalism and neo-Renaissance, placed between traditional historicism and moderate modernity. In other words, the rooted in tradition eclectic dictionary – applied by the architect – became a spedziedzictwa przeszłości a modernizmem. Jak zaznacza D. Watkin, słusznym wydaje się być stwierdzenie, że „Połączenie w nim zmanierowanego wyrafinowanego detalu z czystymi, wyraźnie 'nowoczesnymi' liniami sprawiło, że wywarł on głęboki wpływ na tych architektów międzywojennej Europy, którzy niechętnie przyjmowali styl międzynarodowy", takich jak choćby autor podręcznikowej pracy o tym typie nieawangardowej, tradycjonalnej, a jednocześnie modernistycznej architektury – jak przykładowo rzecz biorąc – Paul Bonatz, twórca dworca kolejowego w Stuttgarcie, wzniesionego w 1911-1927 (Ryc. 30, 31).

Wzniesienie sztokholmskiego Ratusza można przeto uznać za symboliczny moment otwierający nowy rozdział w dziejach sztuki i architektury Danii, Norwegii, Szwecji i Finlandii, który rozsławił powszechnie XX wiek, takimi znakomitościami jak tworzący w duchu zachowawczym Peder Vilhelm Jensen Klint (Katedra Grundtvig, Kopenhaga, 1913-1926), Hack Kampmann (Komenda Główna Policji, Kopenhaga, 1919-1924) lub też łączący dziedzictwo Clode'a Nicolasa Ledoux, Étienne-Nicolasa-Louisa-Davida Boulléego i Jeana-Nicolasa-Louisa-Davida Duranda z nowoczesnością, Erik Gunnar Asplund (Biblioteka Publiczna, 1920-1928, Dom Koncertowy, 1920-1926 – obydwa w Sztokholmie); korelujących tradycję z umiarkowanym racjonalizmem – Kay Fisker, Christian Frederik Møller i Povl Stegmann (kompleks Uniwersytetu w Århus, 1931-1946), podobnie jak Arnstein Arneberg i Magnus Poulsson (Ratusz w Oslo, 1933-1950) oraz zawierające nowy wymiar postępu i stylu międzynarodowego prace takich twórców, jak E.G. Asplund (Pawilon Wystawowy, 1930 i Krematorium, 1935-1940 – Sztokholm) oraz Alvar Aalto (Biblioteka Miejska w Viipuri, 1927-1935, Sanatorium Przeciwgruźlicze w Paimio, 1929-1933, budynek redakcji w Turku, 1927-1929).

Te wspaniałe osiągnięcia dwudziestowiecznej architektury krajów nordyckich przesłoniły dorobek dziewiętnastowiecznej sztuki i budownictwa tego regionu północnej Europy, usuwając w cień nieco zapoznane dziedzictwo neoklasycyzmu, a osobliwie historyzmu i eklektyzmu. Dopiero postmodernizm rozpoczął na nowo odkrywanie artystyczno-estetycznych walorów północnoeuropejskiej architektury XIX wieku i temu problemowi poświęcony został niniejszy esej, mający jednocześnie za zadanie wypełnienie znaczącej luki w polskiej literaturze przedmiotu.

cific *intermezzo* between the culmination of the past heritage and modernism. As D. Watkin points out, it seems justified to claim, that: "The fact it combined mannerist refined detail with pure, clearly 'modern' lines caused it to have a profound impact on those architects in inter-war Europe who reluctantly accepted the international style", such as e.g. the author of the textbook work about this type of non-avant-garde, traditional, and simultaneously modernist architecture, for instance – Paul Bonatz, the designer of the railway station in Stuttgart, erected in the years 1911-1927. (Fig. 30, 31)

Erection of the Town Hall in Stockholm can therefore be regarded as a symbolic moment opening a new chapter in the history of art and architecture in Denmark, Norway, Sweden and Finland, which made the 20th century generally associated with the names of such famous personages as: creating in the conservative spirit – Peder Vilhelm Jensen Klint (Grundtvig Cathedral, Copenhagen, 1913-1926), Hack Kampmann (the Police Headquarters, Copenhagen, 1919-1924) or combining the heritage of Claude Nicolas Ledoux, Étienne-Nicolas-Louis-David Boullée and Jean-Nicolas-Louis-David Durand with modernity, Erik Gunnar Asplund (the Public Library, 1920-1928, the Concert House, 1920-1926 – both in Stockholm); correlating tradition with moderate rationalism – Kay Fisker, Christian Frederik Møller and Povl Stegmann (the University complex in Århus, 1931-1946), similarly to Arnstein Arneberg and Magnus Poulsson (the Town Hall in Oslo, 1933-1950); encompassing the new dimension of progress and international style works of such artists as: E.G. Asplund (the Exhibition Pavilion, 1930 and the Crematorium, 1935-1940 – Stockholm) and Alvar Aalto (the City Library in Viipuri, 1927-1935, the Tuberculosis Sanatorium in Paimio, 1929-1933, the editorial office building in Turku, 1927-1929).

Those magnificent achievements of the twentieth-century architecture in the Nordic countries obscured the achievements of the nineteenth-century art and construction of that region of northern Europe, overshadowing the unacknowledged heritage of neo-classicism, and especially historicism and eclecticism. Only post-modernism began again re-discovering the artistic-aesthetic values of the north-European architecture of the 19th century, and that issue has been the subject of this essay which, at the same time, was meant to fulfil a significant gap in the Polish literature of the subject.

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Streszczenie

Celem zaprezentowanego w tym miejscu eseju jest zapoznanie czytelnika polskiego z, niestety niedocenianą nadal, architekturą krajów skandynawskich i Finlandii w dziewiętnastym stuleciu. Podstawową trudnością niniejszego opracowania pozostaje nader skromna, wręcz uboga, baza źródłowa w literaturze polskiej o neoklasycyzmie, historyzmie i eklektyzmie w tym regionie północnej Europy. Tym problemem interesował się niegdyś Piotr Krakowski, współtwórca, wraz z Zofią Ostrowską-Kębłowską, podwalin wiedzy na temat środkowoeuropejskiego historyzmu i eklektyzmu. Przedmiotowy esej jest przeto hołdem złożonym pamięci Profesora Piotra Krakowskiego, który onegdaj zachęcił niżej podpisanych autorów do wypełnienia znaczącego naukowego niedostatku wiedzy o architekturze tego obszaru kultury europejskiej.

Abstract

The aim of this essay is acquainting the Polish leader with the still not sufficiently appreciated architecture of the Scandinavian countries and Finland in the nineteenth century. The basic difficulty in this study has been an extremely limited, not to say poor, resource base in Polish literature concerning neo-classicism, historicism and eclecticism in that region of northern Europe. The problem was once of interest to Piotr Krakowski who, together with Zofia Ostrowska-Kębłowska, created the foundations of knowledge concerning the central-European historicism and eclecticism. The current essay is therefore a homage paid to the memory of Professor Piotr Krakowski who once encouraged the authors of this article to fill in the significant scientific gap in the knowledge regarding architecture of this area of European culture.

Dominika Kuśnierz-Krupa*

„Portugalska szkoła konserwacji" – Muzeum Transportu i Komunikacji Eduarda Souto Moury w Porto

"Portuguese school of conservation" – Museum of Transport and Communication by Eduardo Souto Moura in Porto

słowa kluczowe: Porto – Alfandega Nova, adaptacja, realizacja konserwatorska, Eduardo Souto Moura

key words: Porto – Alfandega Nova, adaptation, conservation realisation, Eduardo Souto Moura

Porto, drugie co do wielkości miasto w Portugalii, to kolebka wspaniałej, monumentalnej architektury, stąd inicjatywa wpisania centrum miasta na Listę Światowego Dziedzictwa Kulturalnego i Przyrodniczego UNESCO[1].

W 2001 roku Porto zostało wybrane przez Unię Europejską na stolicę kultury, dzięki czemu miało możliwość zaprezentować swoje wartości kulturowe, tradycje oraz najciekawsze obiekty.

Zabytki Porto przedstawiają różnorodne style i pochodzą z wielu epok, od wczesnego średniowiecza po wiek XIX. Prezentują też różną kondycję i stan zachowania substancji zabytkowej. Stąd stosunkowo duży ruch w modernizacji i konserwacji tych obiektów. Niestety zabytki w mieście są tak liczne, że pomimo odczuwalnej na niemal każdym rogu atmosfery wielkiej rewaloryzacji miasta, wciąż można odnaleźć ulice, które pomimo swej malowniczości prezentują bardzo zły stan techniczny[2].

Ruch w konserwacji zabytków w Porto odbija się na ilości nowatorskich rozwiązań i podejściu do modernizacji obiektów zabytkowych. Podobnie jak w Lizbonie, gdzie kunszt miejscowych architektów przy rewaloryzacji takich obiektów jak Muzeum Fado i "Casa dos Bicos" (proj. Santa – Rita Arquitectos); Muzeum „Teatro Romano" (proj. Daniel Ermano); Muzeum Chiado (proj. Jean-Michel Wilmotte), Banco Mais (proj. Gonçalo Byrne Arquitectos) czy Agencia Europeia da Seguranca Maritima (proj. Manuel Tainha)[3] jest nie do podważenia, także w Porto odnaleźć można szczególne i zupełnie wyjątkowe interwencje konserwatorskie.

Na czele tych realizacji znajduje się na pewno projekt Eduarda Souto Moury[4]: modernizacja dawnej komory celnej

Porto, the second largest city in Portugal, is the cradle of magnificent and monumental architecture, hence the initiative of entering the city centre into the UNESCO List of World Cultural and Natural Heritage[1].

In 2001, Porto was chosen by the European Union as its cultural capital, thanks to which it had an opportunity to present its cultural values, traditions and the most interesting objects.

Monuments in Porto represent various styles and originated in various epochs, from the early medieval period until the 19th century. They also represent varying situation and state of preservation of historical substance, hence the relatively intense activity in modernization and conservation of those objects. Unfortunately, monuments in the city are so numerous that despite the atmosphere of intense revalorisation sensed on almost every corner, one can still find streets which, in spite of their picturesqueness, represent very poor technical condition[2].

The flurry of activity in monument conservation in Porto is reflected in the amount of innovative solutions and the approach to modernization of historical objects. Similarly to Lisbon, where the craftsmanship of local architects who restored such objects as: Fado Museum and "Casa dos Bicos" (designed by Santa – Rita Arquitectos); Museum "Teatro Romano" (designed by Daniel Ermano); Chiado Museum (designed by Jean-Michel Wilmotte), Banco Mais (designed by Gonçalo Byrne Arquitectos) or Agencia Europeia da Seguranca Maritima (designed by Manuel Tainha)[3] is indisputable, in Porto one can also find special and absolutely unique examples of conservation intervention.

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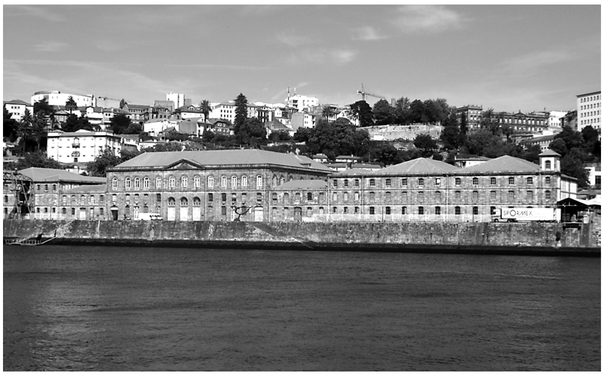
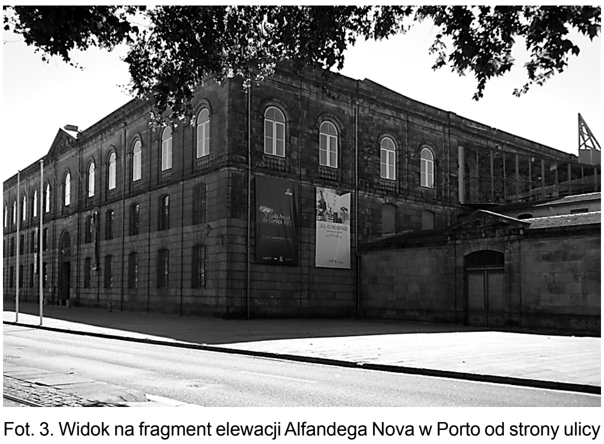


Fot. 1. Widok na budynek Alfandega Nova w Porto z przeciwległego brzegu rzeki Douro, fot. D. Kuśnierz-Krupa, 09.2012
Photo 1. View of the building of Alfandega Nova in Porto from the opposite bank of the river Douro, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 2. Widok na fragment elewacji Alfandega Nova w Porto od strony rzeki Douro, fot. D. Kuśnierz-Krupa, 09.2012
Photo 2. View of the elevation fragment of Alfandega Nova in Porto from the river Douro, photo by D. Kuśnierz-Krupa, 09.2012



Fot. 3. Widok na fragment elewacji Alfandega Nova w Porto od strony ulicy Rua Nova da Alfandega, fot. D. Kuśnierz-Krupa, 09.2012
Photo 3. View of the elevation fragment of Alfandega Nova in Porto from the street Rua Nova da Alfandega, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 4. Widok na fragment elewacji Alfandega Nova, gdzie zastosowano współczesne detale architektoniczne, fot. D. Kuśnierz-Krupa, 09.2012
Photo 4. View of the elevation fragment of Alfandega Nova, where modern architectonic details were used, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 5. Widok na fragment elewacji Alfandega Nova, gdzie zastosowano współczesne detale architektoniczne, fot. D. Kuśnierz-Krupa, 09.2012
Photo 5. View of the elevation fragment of Alfandega Nova, where modern architectonic details were used, photo by D. Kuśnierz-Krupa, 09.2012

Fot.6. Widok na fragment elewacji Alfandega Nova i wejście do budynku, fot. D. Kuśnierz-Krupa, 09.2012
Photo 6. View of the elevation fragment of Alfandega Nova and the entrance to the building, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 7. Budynek Alfandega Nova. Widok na fragment klatki schodowej, fot. D. Kuśnierz-Krupa, 09.2012
Photo 7. The building of Alfandega Nova. View of a fragment of the stairwell, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 8. Budynek Alfandega Nova. Widok na fragment hallu na 1 piętrze, fot. D. Kuśnierz-Krupa, 09.2012
Photo 8. The building of Alfandega Nova. View of a fragment of the hall on 1st floor, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 11, 12. Budynek Alfandega Nova. Widok na współcześnie zaaranżowaną część muzealną, prezentującą historię obiektu, fot. D. Kuśnierz-Krupa, 09.2012
Photo 11, 12. The building of Alfandega Nova. View of a modern arrangement of the museum section, presenting the history of the object, photo by D. Kuśnierz-Krupa, 09.2012

◄ Fot. 10. Budynek Alfandega Nova. Widok na współcześnie zaaranżowaną część muzealną, prezentującą historię obiektu, fot. D. Kuśnierz-Krupa, 09.2012
Photo 10. The building of Alfandega Nova. View of a modern arrangement of the museum section, presenting the history of the object, photo by D. Kuśnierz-Krupa, 09.2012

i adaptacja na Muzeum Transportu i Komunikacji. Budynek znajduje się w dzielnicy Foz, przy Alfandega Rua Nova, nad samym brzegiem rzeki Douro[5]. Historia jego powstania sięga XIX wieku, kiedy to w miarę rozwoju handlu, a co za tym idzie w miarę bogacenia się miasta, lokalna organizacja handlowa Associação Comercial do Porto podjęła decyzję o budowie komory celnej, której architektura podkreśliłaby rangę i prestiż ówczesnego Porto[6].

Nie była to jednak pierwsza siedziba urzędu celnego w mieście. Informacje o poborze opłat celnych w Porto sięgają pierwszej połowy XII wieku[7], skąd można wnioskować, że komora celna istniała w mieście już w średniowieczu. Zresztą w tym okresie kontrola nad dochodami z funkcjonowania portu była zarzewiem konfliktu między Koroną a Biskupstwem[8]. Został on rozstrzygnięty dopiero w 1405 roku przez króla D. Joao I na korzyść Korony. Około osiemdziesięciu lat wcześniej król D. Alfonso IV zainicjował powstanie składu celnego w celu usprawnienia kontroli wpływów z podatków. Skład ten był wielokrotnie przebudowywany w związku ze stale rosnącą liczbą towarów, które były przedmiotem handlu w mieście. Na początku XIX wieku podjęto nawet próby adaptacji na potrzeby komory magazynów zlokalizowanych wzdłuż rzeki. Stało się jednak oczywiste, że zabezpieczenie dochodów z wymiany handlowej na tak dużą skalę wymaga zainwestowania w nowy budynek, który, tak jak wspomniano wyżej, podkreślałby rangę i prestiż miasta, a także zapewniłby odpowiednie zaplecze techniczne [9]. Wybór działki pod przyszłą inwestycję trwał kilka lat. W końcu zadecydowano, że budynek stanie na plaży Miragaia, nad samą rzeką Douro.

Do wykonania projektu zaangażowano znanego w owych czasach francuskiego architekta Jeana F.G. Colsona, który zaprosił do zespołu lokalnych projektantów. W wyniku tej współpracy powstał projekt budynku w stylu neoklasycystycznym, którego architektura nawiązywała do innych obiektów użyteczności publicznej w mieście.

Budowę nowej komory o wysokości 20 m i długości 286 m, rozpoczęto w połowie XIX wieku, by ostatecznie zakończyć w latach 70. Urbaniści i historycy architektury tę właśnie realizację uważają za początek przełomu w krajobrazie kulturowym nabrzeża rzeki Douro w XIX wieku.

Alfandega Nova nie była jednym budynkiem, a całym zespołem obiektów i urządzeń, zaprojektowanym na powierzchni około 36 tys. m^2. W skład kompleksu, prócz budynku głównego, wchodziły także magazyny, linia kolejowa, platformy i inne elementy infrastruktury, niezbędne do funkcjonowania komory.

Konstrukcja obiektu wykorzystuje zarówno żelazo, jak i kamień. W piwnicach stropy oparte są na masywnych kamiennych kolumnach, zaś na wyższych kondygnacjach kolumny wykonano z żeliwa. Jest to jeden z bardziej interesujących przykładów konstrukcji opartej na połączeniu żelaza i kamienia.

Na każdej kondygnacji w posadzkach zamontowano szyny pozwalające na łatwy transport towarów. W miarę jak malało znaczenie transportu wodnego na rzecz lądowego, a później także lotniczego magazyny Alfandega Nova były przenoszone z nabrzeża Douro na terminale kolejowe i lotnicze. Z czasem lokalna organizacja handlowa, zarządzająca budynkiem zdecydowała o przekazaniu go nowopowstałemu stowarzyszeniu Associação para o Museu dos Transportes e Comunicações (A.M.T.C), pozostawiając w nim jedynie część biurową związaną z funkcjonowaniem urzędu celnego.

At the top of the list of those realisations is the project by Eduardo Souto Moura[4]: modernizing the former customs house and converting it into the Museum of Transport and Communication. The building is located in the Foz district, at Alfandega Rua Nova, on the very bank of the river Douro[5]. Its origin dates back to the 19th century when, along with the development of commerce and consequently the city growing richer, a local trade organisation Associação Comercial do Porto decided to have a customs house built whose architecture would emphasise the status and prestige of Porto[6].

It was not the first customs house in the city. Information concerning payment of customs duties in Porto dates back to the first half of the 12th century[7], which leads to the conclusion that a customs house must already have existed in the city during the Middle Ages. At that time the issue of controlling the income from a functioning harbour was the mainspring of conflict between the Crown and the Bishopric[8]. It was only resolved in 1405 by king D. Joao I in favour of the Crown. Around eighty years earlier king D. Alfonso IV initiated establishing a customs house in order to streamline control of tax revenues. The house was rebuilt many times in connection with the constantly growing amount of commodities traded in the city. At the beginning of the 19th century, attempts were made to adapt the storehouses located along the river to the needs of the customs house. However, it soon became obvious, that securing the income from trade exchange on such a large scale required investing into a new building which, as mentioned above, would emphasise the rank and prestige of the city, as well as ensure an appropriate technical base[9]. Selecting a plot for the future investment took several years. Finally, it was decided that the building should be erected on the Miragaia beach, on the river Douro.

A well-known French architect, Jean F.G. Colson, engaged to carry out the project, invited local designers to join his team. This cooperation resulted in designing a building in the neo-classicist style, whose architecture alluded to other public utility objects in the city.

The process of building the new customs house which was 20 m high and 286 m long, commenced in the mid-19th century, and was finally completed in the 1870s. Urban planners and historians of architecture consider this realisation to be the breakthrough in the cultural landscape along the banks of the Douro in the 19th century.

Alfandega Nova was not a single building, but a whole complex of objects and facilities, designed on the area of approximately 36 thousand m^2. Apart from the main building, the complex encompassed warehouses, a railway line, loading platforms and other elements of infrastructure indispensable for the customs house functioning.

Both iron and stone were used for the object construction. Ceilings in cellars were supported by massive stone columns, while on upper storeys columns were made from cast iron. It is one of most interesting examples of a structure based on combining iron and stone.

On each storey rails allowing for easy transport of commodities were fitted in the floor. With the decreasing importance of water transport in favour of land and later air transport, the warehouses of Alfandega Nova were mover from the banks of the Douro to the railway and air terminals. In time, a local trade organisation in charge of the building decided to hand it over to the newly created association Associação para o Museu dos Transportes e Comunicações (A.M.T.C.), leaving there

Fot. 9. Budynek Alfandega Nova. Widok na bibliotekę w części muzealnej na 1 piętrze, fot. D. Kuśnierz-Krupa, 09.2012
Photo 9. The building of Alfandega Nova. View of a library in the museum section on 1st floor, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 15. Budynek Alfandega Nova. Widok na fragment hallu na 2 piętrze, fot. D. Kuśnierz-Krupa, 09.2012
Photo 15. The building of Alfandega Nova. View of a fragment of the hall an 2nd floor, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 13. Budynek Alfandega Nova. Fragment wystroju sali konferencyjnej, fot. D. Kuśnierz-Krupa, 09.2012
Photo 13. The building of Alfandega Nova. Fragment of the decor in the conference room, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 16. Budynek Alfandega Nova. Widok na fragment łącznika zaprojektowanego przez Souto Moure i przejście do drugiej części obiektu, fot. D. Kuśnierz-Krupa, 09.2012
Photo 16. The building of Alfandega Nova. View of a fragment of a walkway designed by Souto Moure and a passage to the other part of the object, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 17. Budynek Alfandega Nova. Widok na fragment łącznika zaprojektowanego przez Souto Moure na tle zabytkowej fasady obiektu, fot. D. Kuśnierz-Krupa, 09.2012
Photo 17. The building of Alfandega Nova. View of a fragment of a walkway designed by Souto Moure against the historical facade of the object, photo by D. Kuśnierz-Krupa, 09.2012

◄ Fot. 14. Budynek Alfandega Nova. Widok na współcześnie zaaranżowaną salę konferencyjną, fot. D. Kuśnierz-Krupa, 09.2012
Photo 14. The building of Alfandega Nova. View of a contemporary arrangement of the conference room, photo by D. Kuśnierz-Krupa, 09.2012

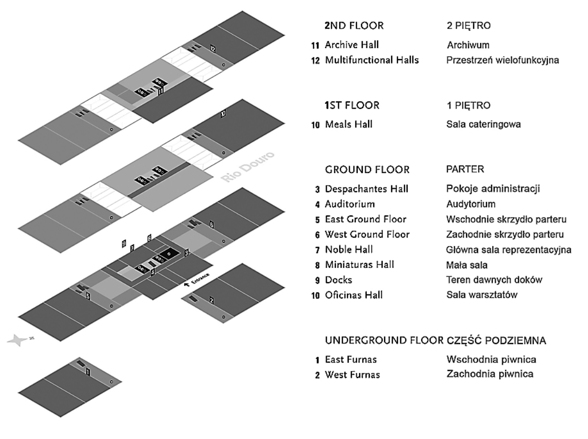
Fot. 18. Budynek Alfandega Nova. Widok na fragment aranżacji części muzealnej – wystawa przedstawiająca początki motoryzacji w Portugalii, fot. D. Kuśnierz-Krupa, 09.2012
Photo 18. The building of Alfandega Nova. View of a fragment of arranged museum space – exhibition depicting the beginnings of motorization in Portugal, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 21. Budynek Alfandega Nova. Widok na fragment przestrzeni muzealnej, zaaranżowanej w piwnicach obiektu, fot. D. Kuśnierz-Krupa, 09.2012
Photo 21. The building of Alfandega Nova. View of a fragment of museum space, arranged in the object cellars, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 19. Budynek Alfandega Nova. Widok na fragment przestrzeni wielofunkcyjnej w poziomie parteru obiektu, fot. D. Kuśnierz-Krupa, 09.2012
Photo 19. The building of Alfandega Nova. View of a fragment of multi-functional space on the ground-floor level, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 22. Budynek Alfandega Nova. Widok na fragment dziedzińca wewnętrznego, fot. D. Kuśnierz-Krupa, 09.2012
Photo 22. The building of Alfandega Nova. View of a fragment of the inner courtyard, photo by D. Kuśnierz-Krupa, 09.2012

Fot. 20. Budynek Alfandega Nova. Fragment przestrzeni wielofunkcyjnej na parterze obiektu, fot. D. Kuśnierz-Krupa, 09.2012
Photo 20. The building of Alfandega Nova. Fragment of multi-functional space on the ground floor, photo by D. Kuśnierz-Krupa, 09.2012



Ryc. 1. Budynek Alfandega Nova. Schemat funkcjonalny, [w:] *Alfandega Congress Centre Porto. Events with Tradition*, Wyd. Edificio de Alfandega, Porto 2011, s. 6
Fig. 1. The building of Alfandega Nova. Functional scheme, [in:] Alfandega Congress Centre Porto. Events with Tradition*, Publ. Edificio de Alfandega, Porto 2011, p. 6.*

W 1993 roku znanemu portugalskiemu architektowi Eduardo Souto de Moura powierzono zadanie stworzenia w tym miejscu Muzeum Transportu i Komunikacji, które prócz funkcji typowo muzealnej pełniłoby też rolę centrum

only the office section connected with the functioning of the customs house.

In 1993, a famous Portuguese architect, Eduard Souto de Moura, was entrusted the task of creating there the Museum of kongresowego z rozbudowaną przestrzenią wielofunkcyjną, która odpowiadałaby na bieżące zapotrzebowanie.

Prace budowlane związane z adaptacją i rewaloryzacją zabytkowego obiektu trwały cztery lata, od 1993 do 1997 roku[10].

Eduardo Souto de Moura przyznał, że praca nad tym projektem przypomniała mu filozofię tworzenia architektury według Aldo Rossiego, który uważał, że „architektura rodzi się z potrzeby, jest niezależna, a w swojej najwyższej formie tworzy przestrzenie muzealne, które dzięki technice mogą się zmieniać i przybierać różne funkcje, które w danym momencie są potrzebne".

Zrewaloryzowany budynek Alfandega Nova posiada duże przestrzenie wielofunkcyjne, zlokalizowane w dwóch symetrycznych skrzydłach, wschodnim i zachodnim. Wszystkie sale są wyposażone w nowoczesny sprzęt audiowizualny oraz systemy oświetlenia pozwalające na realizację w ich wnętrzach rozmaitych wydarzeń[11].

Architekt wprowadzając współczesną funkcję do historycznego budynku wykorzystał istniejącą przestrzeń. Nowe bryły wprowadzone przez projektanta do oryginalnej architektury obiektu to w zasadzie tylko elementy układu komunikacyjnego – łącznik oraz winda wykonane w technologii pozwalającej na jednoznaczną identyfikację granic współczesnej ingerencji architekta.

Efektowne kubatury hallu, korytarzy i sal magazynowych po odrestaurowaniu zostały podkreślone przez dyskretne i wysmakowane nowe detale architektoniczne, jak np. przeszklony wiatrołap przy wejściu głównym. Nowoprojektowane elementy przede wszystkim pozwalają łatwo odczytać oryginalną architekturę budynku, wyraźnie się od niej odcinając, jednocześnie nie próbując jej zdominować, oraz nawiązują do pewnych elementów historycznego składu celnego, takich jak choćby metalowe mechanizmy.

Budynek posiada duże otwory okienne, wpuszczające znaczną ilość światła dziennego. Jednak pokaźny rozmiar pomieszczeń sprawia, że w ich wnętrzach panuje delikatny półcień. Pozwala to na wyeksponowanie niektórych elementów architektury przez wprowadzenie sztucznego oświetlenia. Efekty tego zabiegu widać przede wszystkim w salach konferencyjnych na ostatniej kondygnacji, gdzie udało się w ten sposób podkreślić wspaniałą formę więźby dachowej.

Eduardo Souto de Moura trafnie uznał, że Alfandega Nova jest budynkiem, już w swojej oryginalnej formie, wystarczająco ciekawym i swoje działania sprowadził do podkreślenia jego zalet: wielkich sal magazynowych (nie zaburzając ich kształtu przez wprowadzanie nowych, stałych elementów), pozwalających na swobodną aranżację, zgodną z bieżącymi potrzebami użytkowników; szlachetnych materiałów, z których został wzniesiony – kamienia i stali, ich faktur i kolorów; detali architektonicznych, takich jak stalowe kolumny, kamienne portale czy wspomniana drewniana konstrukcja dachu; oraz elementów stanowiących wyposażenie techniczne składu celnego, jak system szyn stalowych w kamiennych posadzkach, ułatwiających transport składowanych tu niegdyś towarów czy dźwig portowy.

Zaprezentowana realizacja adaptacji dawnej Komory Celnej Alfandega Nova w Porto na Muzeum Transportu i Komunikacji autorstwa Eduardo Souto Moury jest niebanalnym przykładem kunsztu w rewaloryzacji obiektu zabytkowego. Architekt trafnie dobrał współczesne detale architektoniczne i włączył je w dialog z XIX-wieczną formą i materiałem. Zrobił to tak perfekcyjnie, że realizacja ta pomimo upływu lat wciąż wydaje się „świeża" i zgodna z najnowszymi trendami.

Transport and Communication which, besides being a typical museum, would also serve as a congress centre with extended multi-functional space that could satisfy current requirements.

Construction work connected with the adaptation and revalorisation of the historical object lasted four years, from 1993 to 1997[10].

Eduardo Souto de Moura admitted that working on this project reminded him about the philosophy of creating architecture according to Aldo Rossi who believed that "architecture is born out of need, is independent, and in its most superior form creates museum spaces which, thanks to technology, can change and serve various functions regarded as necessary at a given moment".

The restored building of Alfandega Nova boasts of large multi-functional spaces located in two symmetrical wings, eastern and western. All rooms are equipped with modern audio-visual devices and lighting systems allowing for realising various events in those interiors[11].

When introducing a modern function into the historical building, the architect used the already existing space. New features introduced by the designer into the original architecture of the object are merely elements of the communication layout – a walkway and the lift, made with the use of technology allowing for explicitly determining the borderlines of the architect's contemporary interference.

After restoration the stunning volumes of the hall, corridors and storage rooms were highlighted with discreet and refined new architectonic details, such as e.g. a glass porch at the main entrance. First of all, newly-designed elements allow for easily interpreting the original architecture of the building while clearly dissociating themselves from it; though at the same time without trying to dominate it they allude to certain elements of the historical customs house , such as e.g. metal mechanisms.

The building has large windows which let in much daylight. Nevertheless, the immense size of the rooms suffuses the interior with delicate half-light, which allows for emphasising selected elements of architecture by introducing artificial lighting. Effects of that are visible primarily in the conference rooms on the top floor where the method was successfully employed to highlight the magnificent form of roof truss.

Eduardo Souto de Moura aptly observed that Alfandega Nova is an interesting enough building even in its original form, so he narrowed his work to emphasising its merits: enormous storage rooms (not disturbing their shape by introducing new, permanent elements), allowing for arranging them freely in response to the current needs of their users; high-quality materials used for its construction – stone and steel, their texture and colours; architectonic details such as steel columns, stone portals or the already mentioned timber roof truss; as well as elements constituting the technical equipment of a customs house such as a system of steel rails in stone floors facilitating transport of commodities once stored here, or a harbour crane.

The presented adaptation of the former Customs House Alfandega Nova in Porto into the Museum of Transport and Communication, designed by Eduardo Souto Moura, is a remarkable example of artistry in restoration of a historical object. The architect accurately selected modern architectonic details and involved them in a dialogue with the 19th-century form and material. He did it so excellently that, despite the passage of time, the realization still appears "fresh" and in keeping with the latest trends.

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Streszczenie

Artykuł prezentuje adaptacje dawnej komory celnej Alfandega Nova w Porto na Muzeum Transportu i Komunikacji. Autorem projektu tej realizacji konserwatorskiej jest światowej sławy portugalski architekt Eduardo Souto Moura. Obiekt, wzniesiony w latach 70. XIX wieku z inicjatywy organizacji handlowej Associação Comercial do Porto, w 1992 roku został przekazany stowarzyszeniu Associação para o Museu dos Transportes e Comunicações (A.M.T.C.), które zleciło Eduardo Souto Mouro wykonanie projektu adaptacji Alfandega Nova. W zrewaloryzowanym obiekcie miała znaleźć się wysokiej jakości przestrzeń muzealna, a także centrum konferencyjne. Funkcje te zostały niezwykle umiejętnie zaadaptowane w zabytkowych murach XIX-wiecznego obiektu. Architekt w niebanalny sposób wykorzystał większość istniejących detali związanych z poprzednią funkcją Alfandega Nova, takich jak szyny czy żeliwne wyposażenie wnętrz. Dzięki takiemu zabiegowi udało się mu zachować dawny charakter i klimat obiektu pomimo wprowadzenia doń zupełnie odmiennej funkcji.

Abstract

The article presents an adaptation of the former customs house Alfandega Nova in Porto into the Museum of Transport and Communication. The author of the project of this conservation realisation is a world-famous Portuguese architect Eduardo Souto Moura. The object erected in the 1870s on the initiative of a trade organisation Associação Comercial do Porto, in 1992 was handed over to the association Associação para o Museu dos Transportes e Comunicações (A.M.T.C.), which commissioned Eduardo Souto Moura to design an adaptation of Alfandega Nova. The restored object was to house a high quality museum space, as well as a conference centre. Those functions were extremely skilfully adapted within the historical walls of the 19th-century walls. In an original way the architect used the majority of existing details connected with the previous function of Alfandega Nova, such as rails or cast iron fittings. Due to that he succeeded in preserving the former character and ambience of the object despite introducing a completely different function.

Dominika Sarkowicz, Anna Klisińska-Kopacz[1]

Tajemnice „Autoportretu" Henryka Siemiradzkiego – przyczynek do badań nad warsztatem artysty

Secrets of the "Self-portrait" by Henryk Siemiradzki – contribution to the research on the artist's method

Słowa kluczowe: Siemiradzki, autoportret, warsztat, technologia i technika, badania, rentgen, zdjęcia analityczne, FTIR, XRF

Key words: Siemiradzki, self-portrait, technology and technique, research, X-ray, analytical photos, FTIR, XRF

W 2012 roku mija 110 lat od śmierci Henryka Siemiradzkiego (1843–1902).W swoim czasie był jednym z najsławniejszych polskich malarzy, znanym w Polsce i Europie. Dzisiaj powiedzielibyśmy, że był ówczesnym celebrytą, odwiedzanym i przyjmowanym przez osobistości, uroczyście witanym w miastach, do których przybywał. Choć dawny splendor artysty odszedł w zapomnienie, jego dzieła nadal są wysoko cenione. Obecnie znajdują się w licznych muzeach oraz w zbiorach prywatnych. Pojawiają się na aukcjach osiągając wysokie ceny. Najczęściej przedstawiają sceny z życia codziennego starożytnych Rzymian, historii Rzymu oraz z dziejów pierwszych chrześcijan, ilustracje antycznych mitów, sceny biblijne – wątki często pojawiające się w twórczości malarzy – akademików II połowy XIX wieku. To Siemiradzki, jakiego znamy najlepiej.

Spuścizna po mistrzu to także wiele studiów i szkiców, również krajobrazowych, a także portrety. Natomiast (olejne) autoportrety Siemiradzkiego znane są tylko dwa i oba znajdują się w zbiorach Muzeum Narodowego w Krakowie[1]. Pierwszy namalowany został najprawdopodobniej po 1876 r., czyli gdy artysta był mężczyzną w średnim wieku – po 33 roku życia, drugi zaś, u jego schyłku – ok. 1900 roku. Oba są niedokończone, zwłaszcza w partii tła. W przypadku dzieła wcześniejszego, o którym traktuje niniejszy artykuł, mamy do czynienia z malarskim szkicem, porzuconym przez autora w fazie, gdy jego najistotniejsza treść – postać, została zrealizowana. Drugi zaś, *Autoportret z paletą*, z przełomu wieków, wydaje się być po prostu nieukończonym obrazem.

W 2010 roku podjęto konserwację zabezpieczającą autoportretu wcześniejszego, z lat 70. XIX w. Zabiegi były konieczne, aby przygotować obraz do dalekiej podróży do Japonii, gdzie na przełomie 2010 i 2011 roku prezentowano, na kolejnych wystawach, skarby z polskich kolekcji[2]. Działania

In 2012, 110 years have passed since the death of Henryk Siemiradzki (1843–1902). In his time he was one of the most famous Polish painters, known both in Poland and in Europe. Today we would say that he was a celebrity, visited and received by important personages, ceremoniously welcomed on his arrival in cities. Although the past splendour of the artist has fallen into oblivion, his works are still highly appreciated. At present they can be found in numerous museums and private collections. They appear at auctions where they reach high prices. Most frequently they present everyday scenes from the life of ancient Romans, the history of Rome and the history of the first Christians, illustrations for ancient myths, or biblical scenes – themes frequently appearing in the works of painters-academicians in the 2nd half of the 19th century. It is Siemiradzki that we know best.

The legacy of the master involves numerous studies and sketches, also landscapes and portraits. On the other hand, only two (oil) self-portraits by Siemiradzki are known and both are to be found in the collection of the National Museum in Krakow[1]. The first might have been painted after 1876, so when the artist was a middle-aged man – over 33 years old, while the other at the end of his life – around 1900. Both have remained unfinished, particularly in the background part. In the case of the earlier work, which this article concerns, we are faced with a painting sketch abandoned by the author at the stage when its most essential element – the figure, had been realised. The other one, *The Self-portrait with a Palette*, from the turn of the centuries seems to be simply an unfinished painting.

In 2010, protective conservation of the earlier self-portrait, from the 1870s, was carried out. The treatment was necessary in order to prepare the painting for a long trip to Japan where, at the turn of 2010 and 2011, treasures from Polish collections were presented in a series of exhibitions[2]. Conservation activity

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konserwatorskie stały się również impulsem do przeprowadzenia badań nad technologią i techniką wykonania malowidła przyczyniając się w ten sposób do rozpoznania warsztatu artysty. Wyniki przeprowadzonych analiz były zaskakujące i odsłoniły wiele tajemnic, jakie skrywał obraz.

Historia obrazu

„Autoportret" został zakupiony do Muzeum Narodowego w 1965 roku od Leona Siemiradzkiego – syna artysty, w Rzymie na Viale di Villa Massimo 24[3]. Według autorek katalogu zbiorów MNK cz. 1, jest to prawdopodobnie studium do autoportretu zamówionego przez Galerię Uffizi we Florecji[4]. Po sukcesach, jakie odniosły *Pochodnie Nerona* wystawiane w całej Europie (m.in. złoty medal na Wystawie Powszechnej w Paryżu w 1878 r.), sława malarza została ugruntowana, jak również potwierdzona członkostwem w wielu europejskich Akademiach Sztuk Pięknych. Również szacowna galeria postanowiła uhonorować Polaka umieszczając jego autoportret wśród wizerunków znamienitych malarzy. Informację o takim zamówieniu znajdujemy między innymi w biografii artysty, napisanej przez Józefa Dużyka – „Siemiradzki. Opowieść biograficzna"[5], a także w wydanym w 1939 r. katalogu wystawy Siemiradzkiego w Towarzystwie Zachęty Sztuk Pięknych w Warszawie[6]. Wzmiankę, na którą powołuje się Dużyk, o zamówieniu przez Galerię Uffizi autoportretu mistrza, zamieszcza w artykule i wywiadzie z Siemiradzkim niemieckojęzyczny dziennik na Węgrzech „Pester Lloyd" w 1879 r.: „(…) artysta *»zmuszony jest niejako do malowania własnego portretu«* albowiem słynna w świecie Galeria Uffizi we Florencji prosi go o to od dwóch lat"[7]. Należy zwrócić uwagę, że obraz musiał być w takim razie zamówiony w 1877 r. Jak zaznaczają badacze tematu, wspomniany portret nie został odnaleziony, bądź w ogóle nie powstał[8]. Jednakże Galeria Uffizi potwierdza istnienie w archiwach dokumentu świadczącego o takim zamówieniu[9].

Całkiem uzasadnione wydaje się założenie, że omawiane tu studium do autoportretu może właśnie być zaawansowanym, malarskim szkicem do nieodnalezionego obrazu. O tym, że nie jest to praca przewidziana przez autora jako dzieło przeznaczone do galerii, świadczy jego technologia, której szczegóły zostały rozpoznane w trakcie badań obiektu. Zdjęcie w promieniowaniu rentgenowskim ujawniło istnienie jeszcze dwóch kompozycji pod autoportretem!

Stan zachowania obrazu przed konserwacją

Lico obrazu było zabrudzone, werniks nieznacznie pożółkły. Główny problem stanowiły liczne odpryski warstwy malarskiej. Stan techniczny malowidła wynikał głównie z niewłaściwej technologii, czyli nałożenia trzech warstw malarskich należących do różnych kompozycji. Prawdopodobnie spoiwo warstwy wierzchniej (III w. chronologiczna)[10] było chudsze od medium w farbie kompozycji pośredniej (architektonicznej, II w. chronologiczna), ponieważ większość odprysków i odspojeń sytuowała się pomiędzy tymi warstwami. W miejscu starych odprysków i ubytków występowały liczne zmatowiałe retusze.

Marginesy obrazu zawinięte na krosno wszędzie nosiły warstwę malarską. W wielu miejscach były zniszczone, zwłaszcza na krawędziach i w narożnikach. Uszkodzenia powodowały również gwoździe napinające płótno na blejtramie, wbite do połowy długości, a następnie zagięte i dobite do marginesów i krosna.

also gave an impulse to carrying out research concerning the technology and technique employed to execute the painting, thus contributing to identifying the artist's method. Results of carried out analyses were surprising and revealed many secrets concealed by the painting.

History of the painting

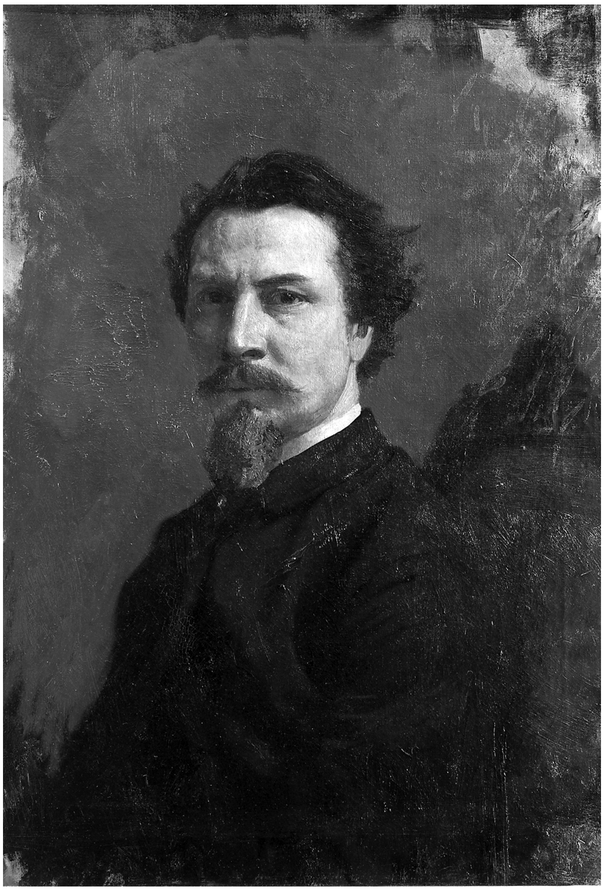
The "Self-portrait" was purchased for the National Museum in 1965, from Leon Siemiradzki – the artist's son, in Rome at 24 Viale di Villa Massimo[3]. According to the authors of the catalogue of the NMK collection, part 1, it is most likely the study for a self-portrait commissioned by the Uffizi Gallery in Florence[4]. After the success of *Nero's Torches* exhibited all over Europe (e.g. awarded the Gold Medal at the World's Fair in Paris in 1878), his position as a painter was consolidated, and confirmed by the membership of numerous European Academies of Fine Arts. An eminent gallery also decided to honour the Pole by placing his self-portrait among the images of other outstanding painters. Information about such a commission has been found in the e.g. biography of the artist written by Józef Dużyk and entitled "Siemiradzki. A biographical story"[5], as well as in the catalogue for an exhibition of Siemiradzki's works at the Society for the Encouragement of Fine Arts in Warszawa, published in 1939[6]. The information which Dużyk refers to, concerning the artist's self-portrait being commissioned by the Uffizi Gallery, was found in an article and an interview with Siemiradzki published in a German-language daily from Hungary "Pester Lloyd" in 1879: "(…) the artist *>is in a way forced to paint his own self-portrait<* as the world – famous Uffizi Gallery in Florence has been asking for it for two years"[7]. It should be noted, that the painting must have been commissioned in 1877. As the experts on the topic have observed, the above mentioned portrait has either not been found, or has never been painted[8]. However, the Uffizi Gallery has confirmed the existence in its archives of a document confirming such a commission[9].

The assumption that the study for a self-portrait, discussed here, could be an advanced painting sketch for a missing painting seems fairly justified. The fact that this work was not intended by the author for a gallery, is confirmed by the technology of the painting the details of which were identified during the examination of the object. The X-ray photograph revealed the existence of two more compositions beneath the self-portrait!

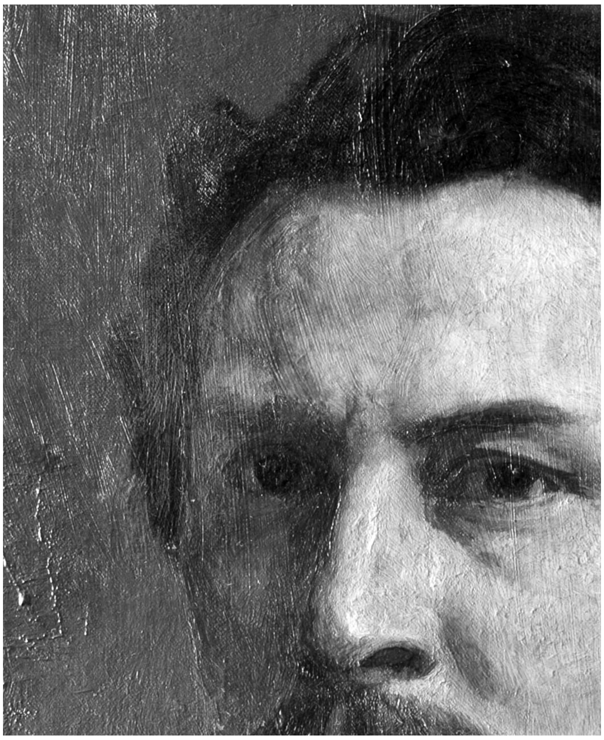
State of preservation of the painting before conservation

The face of the painting was soiled, the varnish was slightly yellowed. The main problem was the flaking off coat of paint. The technical state of the painting resulted mainly from the improper technology i.e. applying three coats of paint belonging to various compositions. The binder of the outer coat (3rd chronological layer)[10] is likely to have been leaner than the medium in the paint of the middle composition (architectonic, 2nd chronological layer), because the majority of flaking off fragments were situated between the two layers. Faded retouches were found in places of old chipped off and missing fragments.

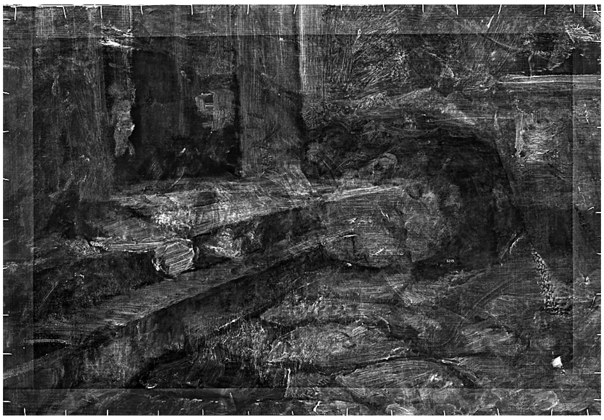
Margins of the painting folded over the canvas stretcher were all covered with a coat of paint. In many places they were damaged, particularly on the edges and in the corners. Damage was also caused by nails tightening the canvas on the stretcher,



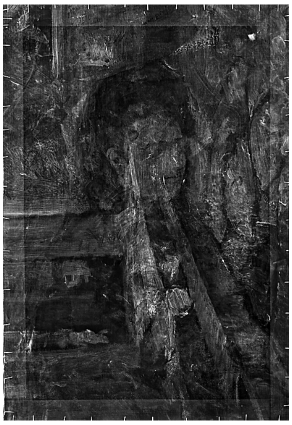
Ryc. 1. Henryk Siemiradzki „Autoportret”, po 1876 r. własność: Muzeum Narodowe w Krakowie
Fig. 1. Henryk Siemiradzki “Self-portrait”, after 1876; property of the National Museum in Krakow



Ryc. 2. Henryk Siemiradzki „Autoportret”, po 1876 r. Fragment lica obrazu – wyraźnie widoczny przebieg śladów szerokiego pędzla, nie pokrywa się z formą malarską widoczną na wierzchu
Fig. 2. Henryk Siemiradzki “Self-portrait”, after 1876. Fragment of the surface – clearly visible traces of a wide brush, do not tally with the painting image visible on the surface

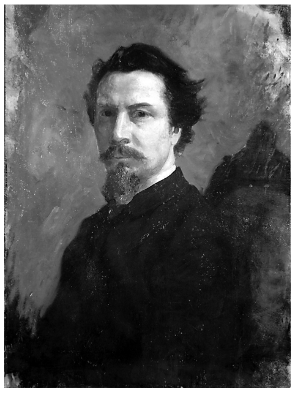


Ryc. 3. Henryk Siemiradzki „Autoportret”, po 1876 r. Rentgenogram obrazu odwrócony o 90° w lewo w stosunku do autoportretu. Widoczna kompozycja architektoniczna
Fig. 3. Henryk Siemiradzki “Self-portrait”, after 1876. X-ray of the painting turned 90° to the left in relation to the self-portrait. Visible architectonic composition



Ryc. 4. Henryk Siemiradzki „Autoportret”, po 1876 r. Rentgenogram obrazu odwrócony o 180° w stosunku do autoportretu. Widoczny szkic głowy kobiety
Fig. 4. Henryk Siemiradzki “Self-portrait”, after 1876. X-ray of the painting turned 180° in relation to the self-portrait. Visible sketch of a woman’s head

Ryc. 5. Henryk Siemiradzki „Autoportret”, po 1876 r. Fragment lica (lewy, górny róg) z widocznym czarnym obramowaniem
Fig. 5. Henryk Siemiradzki “Self-portrait”, after 1876. Fragment of the surface (top left corner) with visible black border



Ryc.7. Henryk Siemiradzki „Autoportret”, po 1876 r. Całość lica w bliskiej podczerwieni. Uwidocznione szczegóły opracowania malarskiego
Fig.7. Henryk Siemiradzki “Self-portrait”, after 1876. The whole surface in near infrared. Visible details of application of paint

Ryc. 6. Henryk Siemiradzki „Autoportret”, po 1876 r. Fragment lica i marginesu (prawy, górny róg). Widoczne zawinięcie malowidła na krosno oraz sposób przybicia gwoździ
Fig. 6. Henryk Siemiradzki “Self-portrait”, after 1876. Fragment of the surface and margin (top right corner). Visible folding of the painting over the stretcher and the way nails were driven in

Wykonane zabiegi konserwatorskie

W podejmowanych zabiegach konserwatorskich zakładano minimalną, konieczną ingerencję w strukturę obiektu, mianowicie zabezpieczenie obrazu bez zdejmowania go z krosna. Nie planowano też usuwania werniksu, ponieważ nie zmieniał on w sposób istotny tonacji malowidła.

Obraz został oczyszczony z zabrudzeń i kurzu, usunięto zmatowiałe retusze. Odspojenia i odpryski podklejano miejscowo przy użyciu Bevy 371 w benzynie, a na zagięciach obrazu na krosna woskiem mineralnym. Rozdarcia płótna na marginesach zreperowano bez zdejmowania z krosna, jedynie miejscowo wyciągając gwoździe i stosując odpowiednie podkładki. W ubytkach wykonano kity (Beva Gesso), na których położono retusze. Lico zabezpieczono werniksem.

Technika i technologia wykonania – badania

Działania konserwatorskie stały się impulsem do podjęcia badań nad technologią i techniką dzieła, co stanowi uzupełnienie niewielkiej jak dotąd wiedzy na temat warsztatu artysty. Przeprowadzono analizę wizualną obrazu oraz wykonano badania pigmentów, zaprawy, płótna i spoiwa warstwy malarskiej. Pobrano próbki do przekrojów bocznych z marginesów zawiniętych na krosno. Szczególnie interesujące wyniki przyniosły zdjęcia w promieniowaniu analitycznym, zwłaszcza w promieniach rentgena. Ponadto malowidło zostało sfotografowane w świetle widzialnym, podczerwieni i ultrafiolecie[11].

Budowa obrazu i technika wykonania na podstawie analizy w świetle białym, w promieniowaniu Rtg, IR, UV oraz obserwacji mikroskopowej

Obraz namalowany jest w technice olejnej na płótnie lnianym (por. wyniki badań). Warstwa zaprawy jest bardzo cienka, z dużą ilością spoiwa, wtarta w płótno. Na przekrojach poprzecznych pobranych próbek, w niektórych miejscach widać, że zaprawa występuje śladowo, głównie w zagłębieniach splotu, pomiędzy nitkami. Płótno jest dość cienkie, o splocie skośnym, gęstość: 9 × 26 nitek na cm^2.

Powierzchnia malowidła, obserwowana w świetle widzialnym, nasuwała podejrzenie, że pod wierzchnią warstwą malarską może istnieć inna kompozycja. W wielu miejscach łatwo było zauważyć niezgodność kierunku i charakteru fakturalnego śladu pędzla, z formą widoczną na powierzchni. Hipoteza ta została potwierdzona dzięki wykonaniu zdjęcia w promieniowaniu rentgenowskim. Co więcej, oprócz warstwy przedstawiającej autoportret odkryto nie jedną, ale dwie kolejne kompozycje.

Na rentgenogramie można odczytać fragment kompozycji architektonicznej występujący bezpośrednio pod portretem własnym artysty. Widać schody prowadzące do szerokich odrzwi, a poniżej kamienny bruk, malowany w sposób tak charakterystyczny dla Siemiradzkiego. Kompozycja ta jest zawarta w formacie prostokąta leżącego. W stosunku do usytuowania autoportretu odwrócona jest o 90°, w lewo.

Przedstawieniem znajdującym się pod kompozycją architektoniczną jest portret kobiety. Ujęta jest w popiersiu, w trzech czwartych, lekko zwrócona w lewo. Na jej głowie widać upięte, dość bujne, ciemne włosy. Portret jest odwrócony o 180° w stosunku do wizerunku malarza. Obie warstwy malarskie uwidocznione na rentgenogramie

stuck half-way their length and then bent and drove in to the margins and the stretcher.

Carried out conservation treatment

The undertaken conservation procedures assumed the minimum necessary interference into the object structure, namely conserving the painting without taking it off the stretcher. Removal of varnish was not planned either, since it did not significantly change the colour scheme of the painting.

The painting was cleaned from dirt and dust, faded retouches were removed. Chipped off and flaking off fragments were glued in places using Bevy 371 in petrol, and on the folds of the canvas onto the stretcher using mineral wax. Torn canvas on the margins was repaired without taking it off the stretcher, only by removing nails in places and applying suitable pads. Missing fragments were filled in with putty (Beva Gesso), on which retouches were made. The face was protected with varnish.

Technique and technology of execution – research

Conservation activity also gave an impulse to carrying out research concerning the technology and technique employed to execute the painting, which has contributed to complementing the little knowledge there had been concerning the artist's method. A visual analysis of the painting was carried out and the pigments, ground, canvas and binder of the coat of paint were carefully examined. Samples for side sections were taken from margins folder over the stretcher. Particularly interesting results were obtained from the photographs in analytical radiation, and especially X-ray photos. Moreover, the painting was photographed in visible light, infrared and ultra-violet[11].

Structure of the painting and the technique of its execution on the basis of the analysis in white light, X-ray, IR, UV and microscope observation

The portrait was painted with oil paint on linen canvas (see test results). The ground layer is very thin with a large amount of binder, rubbed into the canvas. The cross sections of taken samples show that in some places, mainly in the hollow between the threads of the weave, there are merely traces of ground. The canvas is fairly thin, with twill weave, density: 9 × 26 threads in cm^2.

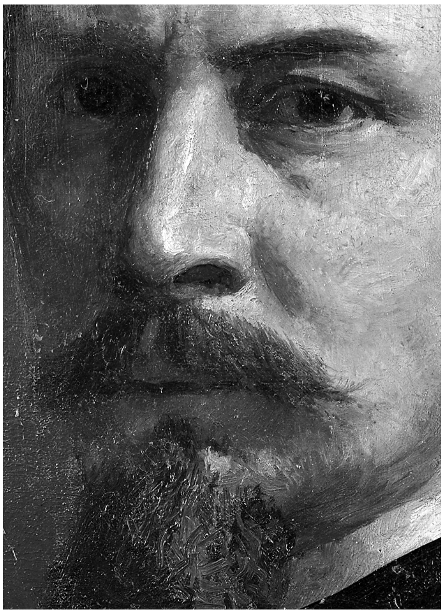
The surface of the painting, observed in visible light, suggested a suspicion that another composition might exist under the outer coat of paint. In several places it was easy to notice an inconsistency between the direction and character of the textural brushstrokes and the form visible on the surface. The hypothesis was confirmed by taking an X-ray photograph. Moreover, not just one, but two subsequent compositions were discovered beneath the layer presenting the self-portrait.

In the X-ray photograph one can discern a fragment of an architectonic composition appearing directly beneath the self-portrait of the artist. A staircase leading towards a wide door can be seen, with cobblestones below, painted in the way so characteristic for Siemiradzki. The composition is encompassed in the format of a lying rectangle. In relation to the self-portrait the composition is turned left by 90°.

The image found beneath the architectonic composition is the portrait of a woman. It is presented as a bust, in three quarters, turned slightly left. Fairly luxuriant, dark hair can be

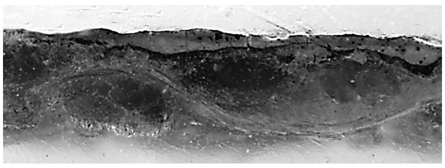
Ryc. 8. Henryk Siemiradzki „Autoportret”, po 1876 r. Fragment lica w promieniowaniu UV. Widoczny sposób opracowania malarskiego karnacji krótkimi pociągnięciami drobnego pędzla. Zaciemnienie fluorescencji wynika prawdopodobnie z obecności pigmentów żelazowych. Nie są to z pewnością retusze

Fig. 8. Henryk Siemiradzki “Self-portrait”, after 1876. Fragment of the surface in UV light. Visible manner of applying paint with short strokes of a fine brush to achieve complexion. Darkened fluorescence resulted probably from the presence of iron pigments. These are certainly not retouches



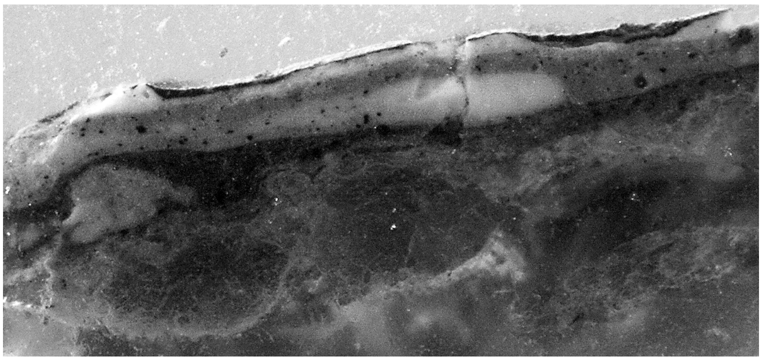
Ryc. 9. Henryk Siemiradzki „Autoportret”, po 1876 r. Fragment lica w świetle vis. Widoczny sposób opracowania malarskiego – drobne, gęste pociągnięcia małego pędzla

Fig. 9. Henryk Siemiradzki “Self-portrait”, after 1876. Fragment of the surface in the visible light. Noticeable manner of applying paint – with small, dense strokes of a fine brush



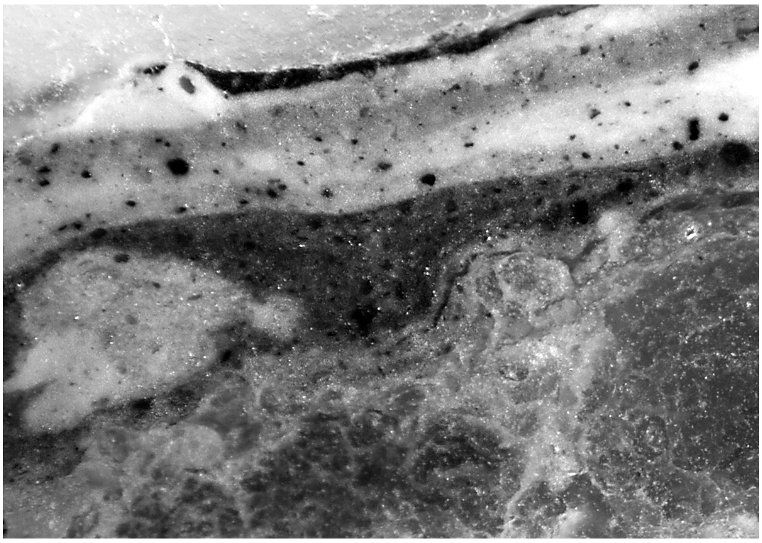
Ryc. 10. Henryk Siemiradzki „Autoportret”, po 1876 r. Zdjęcie mikroskopowe przekroju poprzecznego warstw technologicznych próbki nr 7. Widać, że miejscami zaprawa występuje śladowo

Fig. 10. Henryk Siemiradzki “Self-portrait”, after 1876. Microscope photo of the cross-section of technological layers from sample no 7. It can be seen that only traces of primer occur in places



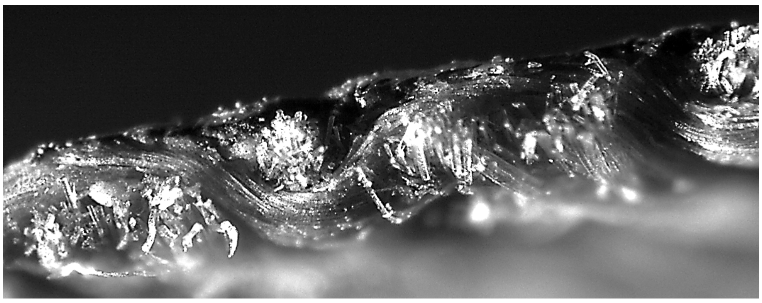
Ryc. 11. Henryk Siemiradzki „Autoportret”, po 1876 r. Zdjęcie mikroskopowe przekroju poprzecznego warstw technologicznych próbki nr 5. Widoczna cienka zaprawa z dużą ilością spoiwa i nakładające się warstwy malarskie

Fig. 11. Henryk Siemiradzki “Self-portrait”, after 1876. Microscope photo of the cross-section of technological layers from sample no 5. Visible thin primer with a large amount of binder and overlapping coats of paint



Ryc. 12. Henryk Siemiradzki „Autoportret”, po 1876 r. Zdjęcie mikroskopowe przekroju poprzecznego warstw technologicznych próbki nr 5; fragment. Widać: bardzo cienką warstwę zaprawy wypełniającej zagłębienia pomiędzy nitkami płótna; warstwę malarską I (szarozieloną z wtrąceniami drobin innych pigmentów); warstwę malarską II (biel złamaną innym kolorem, w trzech warstwach, malowaną „mokre w mokre”, błękit, biel); warstwa malarska III (czerń). Ciemna, cienka linia w I warstwie malarskiej nie pojawia się w przekrojach innych próbek. Dlatego trudno ją interpretować jako przeklejenie czy izolację. Możliwe, że jest to fragment podrysowania / podmalowania

Fig. 12. Henryk Siemiradzki “Self-portrait”, after 1876. Microscope photo of the cross-section of technological layers from sample no 5; fragment. Visible: a very thin layer of primer filling in hollows between the threads of the canvas; coat of paint I (grey-green with inclusions of specks of other pigments); coat of paint II (white toned down with another colour, in three layers, painted “wet on wet”, blue, white); coat of paint III (black). The dark thin line in I coat of paint does not occur in cross sections of other samples. Therefore it is difficult to interpret it as saturating or insulation. It may be a fragment of a pencil sketch / underpainting



Ryc. 13. Henryk Siemiradzki „Autoportret”, po 1876 r. Zdjęcie mikroskopowe przekroju poprzecznego warstw technologicznych próbki nr 4 (nie zatopiona w żywicy). Widać przekrój boczny marginesu obrazu. Zaprawa prawie niewidoczna, jedynie w zagłębieniu między nitkami zaznaczonym strzałką

Fig. 13. Henryk Siemiradzki “Self-portrait”, after 1876. Microscope photo of the cross-section of technological layers from sample no 4 (not sunk in resin). Visible side section of the painting margin. Primer almost invisible, except in the hollow between the threads, marked with an arrow

sprawiają wrażenie raczej swobodnie traktowanych szkiców studyjnych.

Tak więc Siemiradzki namalował swój portret na podobraziu, którego już wcześniej użył do celów studyjnych. Pracę rozpoczął na większym kawałku zamalowanego już płótna, jak się wydaje, nie napiętego na krosno. Tego typu luźne, nie napięte na blejtram, szkice olejne na płótnie, które artysta często wykonywał przygotowując się do większych kompozycji, można znaleźć również w zbiorach MNK. W trakcie pracy autor dopasował malowany obraz do krosien. Kadrując go, wykreślił dwie krawędzie czarną farbą ograniczając w ten sposób zakres kompozycji. Następnie zagiął marginesy malowidła na listwach krosna, dlatego noszą one warstwy malarskie. Być może obraz nie został napięty na krosno przez samego autora, jednak nie ulega

seen pinned up on her head. The portrait is turned by 180° in relation to the image of the painter. Both coats of paint visible in the X-ray photo give an impression of fairly freely treated study sketches.

Therefore, Siemiradzki painted his self-portrait on the underpainting which he had previously used for study purposes. He began the work on a larger piece of the already painted canvas, apparently not stretched on the wooden stretcher. That type of loose oil sketches on canvas not tightened on the stretcher, which the artist frequently made while preparing for larger compositions, can also be found in the collection of the NMK. During the work the author fitted the painted picture to the stretcher. Cropping it, he drew two edges with black paint thus limiting the range of the composition. Then he folded the margins of the painting over the stretcher, that is why they

Tabela 1. Uproszczona stratygrafia obrazu
Table 1. Simplified stratygraphy of the painting

Nr	Oznaczenie graficzne warstw technologicznych *Graphic marking of technological layers*	Warstwa chronologiczna *Chronological layer*	Datowanie *Dating*	Określenie warstwy *Layer identification*
1.		V	XX w.	retusze *retouches*
2.		IV	?	werniks *varnish*
3.		III	Po 1876 *After 1876*	gwoździe *nails*
4.		II		warstawa malarska – Autoportret *coat of paint – Self-portrait*
5.				warstwa malarska Kompozycja architektoniczna (?) *coat of paint Architectonic composition* (?)
6.		I		warstwa malarska – Głowa kobiety *coat of paint – Head of a woman* (?)
7.				zaprawa *primer*
8.				płótno *canvas*
9.		I (?)		krosno *stretcher*

Opis warstw technologicznych
(1, V) retusze
(2, IV) werniks (nie autorski ?)
(3,III) gwoździe
(4, III) warstwa malarska olejna autorska – autoportret
(5, II) warstwa malarska olejna autorska – kompozycja architektoniczna
(6, I) warstwa malarska olejna autorska – głowa kobiety
(7, I) zaprawa
(9, I) płótno lniane
(10,I) krosno drewniane

Description of technological layers
(1, V) retouches
(2, IV) varnish (not author's ?)
(3,III) nails
(4, III) author's coat of oil paint – self-portrait
(5, II) author's coat of oil paint – architectonic composition
(6, I) author's coat of oil paint – head of a woman
(7, I) primer
(9, I) linen canvas
(10,I) wooden stretcher

wątpliwości, że czarna linia wyznaczająca zakres kompozycji została wykonana przez artystę i było to elementem dość spontanicznego procesu twórczego. Potwierdzać to może fakt występowania czerwieni z tła właśnie na czerni „ramki". Również analiza składu pierwiastkowego czerni z obramowania i z innych partii kompozycji potwierdza tę tezę. Obraz napięto na drewniane krosno o łączeniu prostym, czopowym, z dwoma klinami. Użyto długich gwoździ, których nie wbijano do końca, a wystającą część zaginano do listwy krosna. Miało to zapewne służyć lepszemu przymocowaniu płótna. Marginesy przycięto do grubości listwy krosna.

Zdjęcia w promieniowaniu ultrafioletowym i podczerwonym również uwidoczniły pewne szczegóły opracowania malarskiego w warstwie autoportretu. W tle, pod czerwoną, jaśniejszą warstwą farby, widać w podmalowaniu swobodne pociągnięcia szerokiego, płaskiego pędzla, z użyciem ciemnej czerwieni, miejscami wpadającej w brąz, a nawet czerń. Prawdopodobnie autor w ten sposób zamalował kompozycję architektoniczną robiąc tym samym podkład pod studium portretowe. W partii tła oraz ubrania malarz użył dużych pędzli o szerokości: 4 cm, 3,5 cm oraz 1,7 cm. Zupełnie inny jest sposób opracowania szczegółów twarzy, włosów, brody. Zarówno w świetle widzialnym jak i w ultrafiolecie (w każdym w inaczej) dostrzec można gęsto nakładane, nieco rozwibrowane, krótkie pociągnięcia drobnego pędzla. Można zidentyfikować pędzle o szerokości: 2, 3 i 4 mm. Artysta dopracowuje szczegół, równocześnie jednak nie obawia się pozostawienia faktury i śladu narzędzia. Wydaje się, że niejednokrotnie stosował te środki celowo. Fragmenty ubrania traktował swobodniej, aby w partii tła dać upust swemu malarskiemu temperamentowi. Oparcie krzesła za plecami portretowanego, to zaledwie zasugerowanie kształtu. Można uznać, że obraz nie jest dokończony, a artysta porzucił pracę nad nim w momencie dopracowania szczegółów samego portretu.

Na licu występuje werniks, prawdopodobnie nie autorski. Jego zakres nie pokrywa się z powierzchnią lica – zapewne założony został później, w świetle ramy.

Analiza materiałów i technologii na podstawie badań

Próbki do obserwacji mikroskopowej przekrojów bocznych warstw technologicznych pobierano jedynie z marginesów założonych na krosno. Badania analityczne metodami spektralnymi (XRF, FTIR) wykonano w Laboratorium Analiz i Nieniszczących Badań Obiektów Zabytkowych Lanboz Muzeum Narodowego w Krakowie.

Analizę składu pierwiastkowego pigmentów przeprowadzono nieinwazyjną metodą – spektroskopii fluorescencji rentgenowskiej, **XRF** (aparatem ArtTAX, Bruker).

Pomiary wykonano w 20 punktach, przy następujących parametrach: źródło wzbudzenia – lampa Rh, U 50 kV, pomiar w atmosferze powietrza, czas akumulacji widma 300 sekund.

Wniosek: w badanych punktach wykryto pierwiastki wskazujące na obecność: bieli ołowiowej (Pb(CO3)2 · Pb(OH)2), bieli cynkowej (ZnO), vermilionu (HgS), naturalnych pigmentów żelazowych (Fe2O3) z domieszkami mineralnymi, czerni żelazowej (Fe3O4), żółcieni neapolitańskiej (Pb3(SbO4)2), błękitu kobaltowego (CoAl2O4), żółcieni chromowej (PbCrO4). żółcieni kadmowej (CdS + BaSO4), błękitu pruskiego KFe[Fe(CN)6].

are covered with coats of paint. The painting might not have been mounted on the stretcher by the author himself, however, there is no doubt that the black line marking the range of the composition was drawn by the artists and it was an element of a fairly spontaneous creative process. It seems to be confirmed by the fact of the red from the background occurring just on the black of the "frame". Also the analysis of the chemical content of the black from the frame and other parts of the composition seems to confirm that thesis. The painting was mounted on a wooden stretcher joined using a mortise and tenon, with two wedges. Long nails were used, which were not driven in to their full length, and their protruding part was bent to the stretcher slat. It might have served to secure the canvas better. Margins were cut to the width of the stretcher slat.

The photographs taken in ultra-violet and infrared radiation also made visible certain details of the painter's technique in the self-portrait layer. In the background, under the red, fighter layer of paint, free strokes of a wide, flat brush are visible in the grounding, using dark red verging on brown or even black in places. It is likely that in this way the author painted over the architectonic composition thus creating the basis for a portrait study. In the section of the background and the clothes, the painter used large brushes 4 cm, 3.5 cm, and 1.7 cm wide. The way of painting the details of the face, hair and beard was completely different. Both in the visible light and in ultra-violet (differently in each case) densely applied, slightly vibrant, short strokes of a fine brush can be discerned. Brushes 2, 3 and 4 mm wide can be identified. The artist worked on every detail, but at the same time he was not afraid to leave the texture and traces of his tool. He seems to have repeatedly used those measures on purpose. Fragments of attire were treated more freely, so that in the part of the background he could let go of his painter's temper. The back of the chair behind the portrayed is merely a suggestion of a shape. It can be assumed that the painting is unfinished, and the artist abandoned work on it at the moment when the details of the portrait itself had been polished.

On the face there is varnish, probably not applied by the author. Its range does not cover the whole surface – it might have been applied later, to the framed painting.

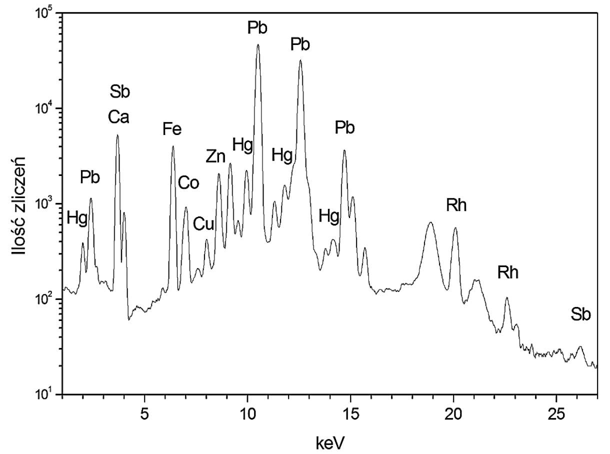
Analysis of materials and technology on the basis of research

Samples for microscopic observation of side sections in technological layers were taken only from the margins folder over the stretcher. Analytical examination using spectral methods (XRF, FTIR) carried out in the Laboratory of Analyses and Non-destructive Examination of Historical Objects Lanboz at the National Museum in Krakow.

Analysis of the chemical content of pigments was conducted using the non-invasive method of X-ray fluorescence spectrometer, **XRF** (with ArtTAX, Bruker).

The measurements were taken in 20 points, with the following parameters: excitation source – Rh lamp, U 50 kV, measurement taken in the air atmosphere, time of spectre accumulation 300 seconds.

Conclusion: the examined spots revealed elements indicating the presence of: lead white (Pb(CO3)2 · Pb(OH)2), zinc white (ZnO), vermilion (HgS), natural iron pigments (Fe2O3) with mineral additions, iron black (Fe3O4), Naples yellow (Pb3(SbO4)2), cobalt blue (CoAl2O4), chrome yellow (PbCrO4), cadmium yellow (CdS + BaSO4), Prussian blue KFe[Fe(CN)6].

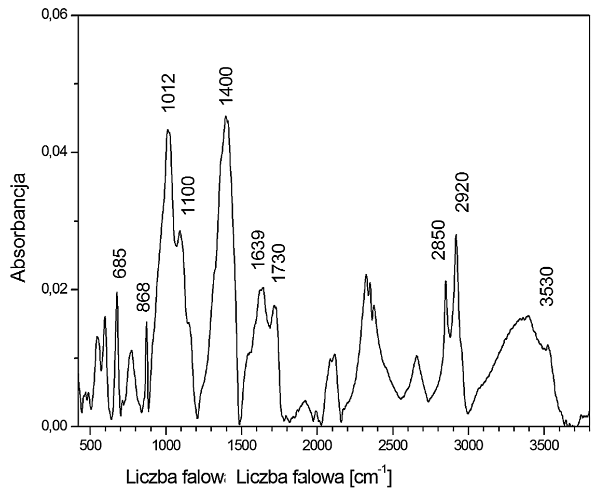


Wykres 1. Wynik analizy spektralnej fluorescencji rentgenowskiej warstwy malarskiej w miejscu czarnego obramowania: Zn, Pb, Fe, Ca, Co, Hg, Sb

Wykonano również analizę próbki (nr 3) warstwy malarskiej wraz z zaprawą pobranej z fragmentu uszkodzonego marginesu obrazu zawiniętego na krosno, oraz nitki z płótna stosując metodę spektroskopii w podczerwieni spektrometrem **FTIR** (IR-Affinity, Shimadzu) pracującym w zakresie 400 – 4000 cm^{-1}, z rozdzielczością 4 cm^{-1}. Widoczne pasma zostały zidentyfikowane jako:

– próbka nr 3. – warstwa malarska z zaprawą:

- 685 cm^{-1} rozciągające CO32 – (kreda)
- 720 cm^{-1} skręcające C-H (klej glutynowy)
- 868 cm^{-1} rozciągające CO32 – (kreda lub klej glutynowy)
- 1012 cm^{-1} rozciągające C-O (olej lniany?)
- 1100 cm^{-1} rozciągające CO32 – (kreda)
- 1400 cm^{-1} rozciągające CO32 – (kreda)
- 1639 cm^{-1} rozciągające C-C (czerń węglowa)
- 1650 cm^{-1} rozciągające C-N-H (klej glutynowy)
- 1730 cm^{-1} rozciągające C=O (olej lniany)
- 2850 cm^{-1} rozciągające C-H (olej lniany)
- 2920 cm^{-1} rozciągające C-H (olej lniany)
- 3547 cm^{-1} rozciągające O-H.



Wykres 2. Wynik analizy próbki (nr 3) warstwy malarskiej wraz z zaprawą metodą spektroskopii w podczerwieni spektrometrem FTIR. Pasma absorbancji potwierdzające obecność: czerni węglowej, oleju lnianego, kredy, kleju glutynowego (?)

Wniosek: w analizowanej próbce wykryto obecność organicznej czerni węglowej, a także oleju jako spoiwa farby.

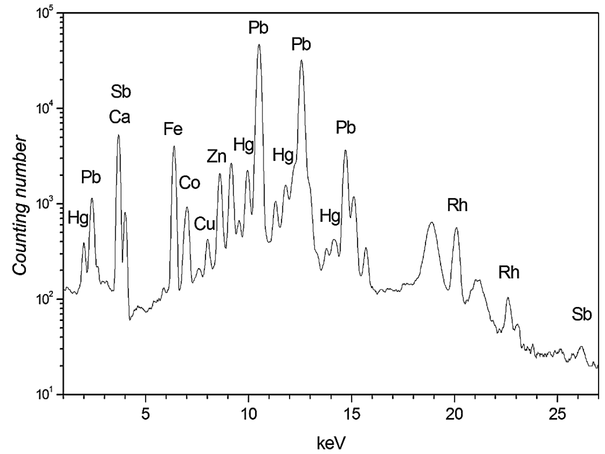


Diagram 1. Results of spectral analysis of X-ray fluorescence of the coat of paint in the black border: Zn, Pb, Fe, Ca, Co, Hg, Sb

An analysis of sample (no 3) of coat of paint with the primer taken from the fragment of a damaged canvas margin folder over the stretcher was also carried out, as well as of a thread from the canvas, using the method of infrared spectroscopy with a **FTIR** spectrometer (IR-Affinity, Shimadzu) operating within the range of 400 – 4000 cm^{-1}, and the resolution of 4 cm^{-1}. Visible bands were identified as:

– sample no 3. – coat of paint with the primer:

- 685 cm^{-1} stretching CO32 – (chalk)
- 720 cm^{-1} twisting C-H (gluten glue)
- 868 cm^{-1} stretching CO32 – (chalk or gluten glue)
- 1012 cm^{-1} stretching C-O (linseed oil?)
- 1100 cm^{-1} stretching CO32 – (chalk)
- 1400 cm^{-1} stretching CO32 – (chalk)
- 1639 cm^{-1} stretching C-C (coal black)
- 1650 cm^{-1} stretching C-N-H (gluten glue)
- 1730 cm^{-1} stretching C=O (linseed oil)
- 2850 cm^{-1} stretching C-H (linseed oil)
- 2920 cm^{-1} stretching C-H (linseed oil)
- 3547 cm^{-1} stretching O-H.

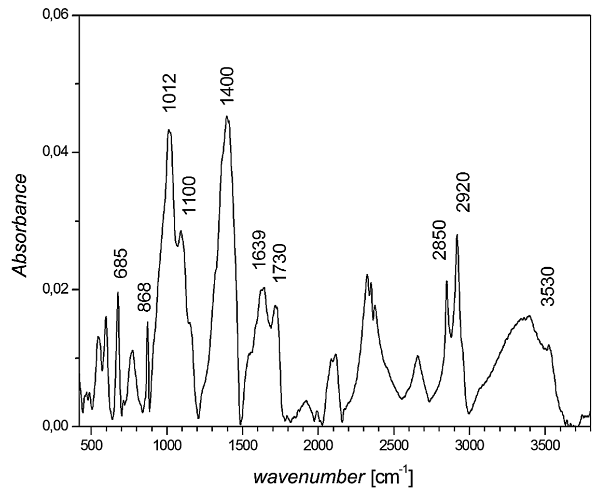


Diagram 2. Results of the analysis of sample (no 3) of coat of paint with primer using the method of infrared spectroscopy with the FTIR spectrometer. Absorbance bands confirmed the presence of: coal black, linseed oil, chalk, gluten glue (?)

Conclusion: the presence of organic coal black and of oil used as paint binder was discovered in the analysed sample.

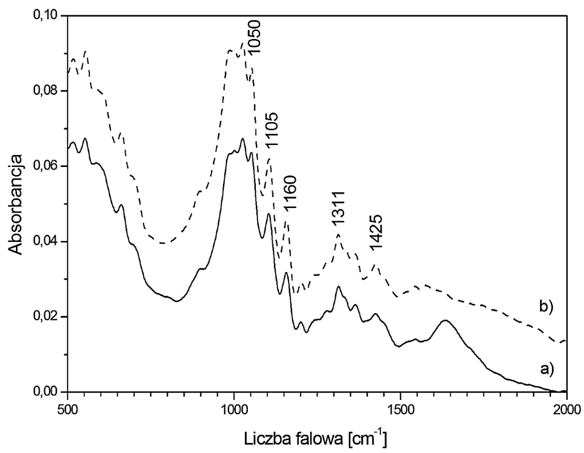
W skład zaprawy wchodzi kreda oraz klej glutynowy. Trudno stwierdzić, czy olej pochodzi wyłącznie od warstwy malarskiej czy znajduje się również w zaprawie.

– próbka nr 8 – warstwa malarska (czerwień), zaprawa:
- 910 cm^{-1} rozciągające C-O (kraplak)
- 1035 cm^{-1} rozciągające C-O (kraplak)
- 1100 cm^{-1} rozciągające C-O (kraplak)
- 1396 cm^{-1} drgania zginające C-H typowe dla zwiazków olejowych bądź żywicznych
- 1530 cm^{-1} (pasmo charakterystyczne dla drgań wiązań aromatycznych)
- 2850 i 2920 cm^{-1} drgania rozciągające C-H typowe dla związków olejowych bądź żywicznych
- 3400 cm^{-1} (O-H drgania rozciągające) (kraplak)

Wniosek: w analizowanej próbce stwierdzono obecność barwnika organicznego z grupy kraplaków, olej.

– nitka:
- 1050 rozciągające C-OH
- 1105 rozciągające C-O-C
- 1160 rozciągające C-C
- 1311 rozciągające C-H
- 1425 rozciągające C-H

– drgania rozciągające dla włókien celulozowych – len.



Wykres 3. Wynik analizy nitki z płótna podobrazia metodą spektroskopii w podczerwieni spektrometrem FTIR. Pasma absorbancji odpowiadające włóknom celulozowym.

Wnioski: Obraz namalowano w technice olejnej[12] na płótnie lnianym. Zaprawa prawdopodobnie emulsyjna (klejowa, z dodatkiem oleju), z kredą jako wypełniaczem. Ponieważ pasma oleju pochodzą również z farby, jego obecność w zaprawie nie jest jednoznaczna. Jednak porównanie analizy „Autoportretu" do wyników badań zaprawy w innych szkicach Siemiradzkiego potwierdza wniosek, że mamy tu do czynienia z zaprawą kredową z klejem glutynowym i olejem lnianym. Dotychczas pięć szkiców artysty ze zbiorów MNK zostało zbadanych przez panią Katarzynę Pakułę w ramach pracy magisterskiej[13]. Oprócz „Tancerki na linie" wyróżniającej się całkiem nietypową technologią, w pozostałych czterech pracach w zaprawie stwierdzono obecność oleju lnianego.

The primer contains chalk and gluten glue. It is difficult to state whether the oil came solely from the coat of paint or whether it was the ingredient of the primer, too.

– sample no 8 – coat of paint (red), primer:
- 910 cm^{-1} stretching C-O (rose madder)
- 1035 cm^{-1} stretching C-O (rose madder)
- 1100 cm^{-1} stretching C-O (rose madder)
- 1396 cm^{-1} bending vibrations C-H typical for oil or resin compounds
- 1530 cm^{-1} (band characteristic for vibrations of aromatic bonds)
- 2850 i 2920 cm^{-1} stretching vibrations C-H typical for oil or resin compounds
- 3400 cm^{-1} (O-H stretching vibrations) (rose madder)

Conclusion: the presence of an organic pigment from rose madder group was discovered in the analysed sample – red, crimson, oil.

– the thread:
- 1050 stretching C-OH
- 1105 stretching C-O-C
- 1160 stretching C-C
- 1311 stretching C-H
- 1425 stretching C-H

– stretching vibrations for cellulose fibres – linen.

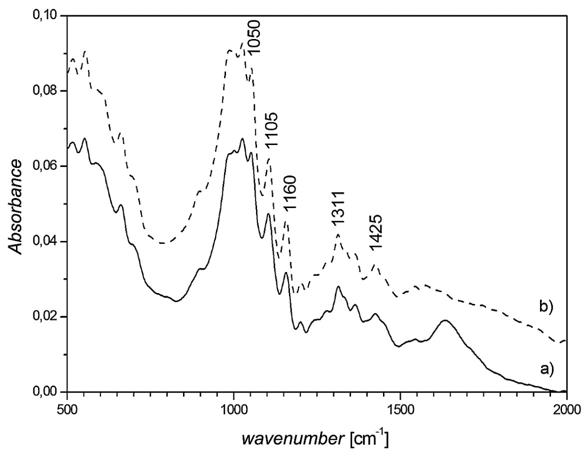


Diagram 3. Analysis results for the thread from the canvas underpainting, using the method of infrared spectroscopy with the FTIR spectrometer. Absorbance bands correspond to cellulose fibres.

Conclusions: The image was painted in oil[12] on linen canvas. Most likely it was an emulsion primer (glue, with oil addition), with chalk as the fill-in. Because oil bands came also from the paint, its presence in the primer is ambiguous. However, comparing the analysis of "Self-portrait" with the results of primer analysis in other sketches by Siemiradzki seems to confirm the conclusion that we are dealing here with chalk primer with gluten glue and linseed oil. So far, five sketches of the artist found in the collection of the NMK have been examined by Ms. Katarzyna Pakuła for her M.A. thesis[13]. Apart from "The Tightrope Dancer" distinguished by its unique technology, the presence of linseed oil has been confirmed in the primer in the four remaining pieces.

Zestawienie wyników badań

Tabela 2. Zestawienie wyników badań spektralnych

Warstwa technologiczna	Wyniki badań	Metoda badań
podobrazie	len	FTIR
zaprawa	Kreda ($CaCO_3$), Klej glutynowy , olej (?)	XRF, FTIR
Warstwa malarska (4, III) Wykryte pierwiastki mogą również pochodzić z warstw malarskich (6, I) i (5, II)	biel ołowiowa ($Pb(CO_3)_2 \cdot Pb(OH)_2$), biel cynkowa ($ZnO$), vermilion ($HgS$), naturalne pigmenty żelazowe (Fe_2O_3) + domieszki mineralne, czerń żelazowa (Fe_3O_4), żółcień neapolitańska ($Pb_3(SbO_4)_2$), błękit kobaltowy ($CoAl_2O_4$), żółcień chromowa ($PbCrO_4$). żółcień kadmowa ($CdS + BaSO_4$), błękit pruski $KFe[Fe(CN)_6]$.	XRF
	czerń węglowa (głównie C), kraplak, olej	FTIR

Results of analytical tests

Table 2. Results of the spectral analysis

Technological layer	Test results	Test method
underpainting	linen	FTIR
primer	chalk ($CaCO_3$), gluten glue (?), oil (?)	XRF, FTIR
coat of paint (4, III) discovered elements might also have come from coats of paint (6, I) and (5, II)	lead white ($Pb(CO_3)_2 \cdot Pb(OH)_2$), zinc white ($ZnO$), vermilion ($HgS$), natural iron pigments (Fe_2O_3) + mineral additions, of iron black (Fe_3O_4), Naples yellow ($Pb_3(SbO_4)_2$), cobalt blue ($CoAl_2O_4$), chrome yellow ($PbCrO_4$), cadmium yellow ($CdS + BaSO_4$), Prussian blue $KFe[Fe(CN)_6]$.	XRF
	coal black (mainly C) oil	FTIR

Podsumowanie

Przeprowadzone badania pozwoliły rozpoznać technologię i technikę warsztatu zastosowaną przez Siemiradzkiego w „Autoportrecie". Ich wyniki ostatecznie wykluczyły, aby było to nieodnalezione, jak do tej pory, dzieło malowane dla Galerii Uffizi. Prawdopodobnie jest to jedynie zaawansowane studium do niego. Niemniej, badany obraz pozostaje jedną z najlepszych prac portretowych artysty ukazując rys psychologiczny oraz temperament malarski autora.

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Autor badań analitycznych: dr Anna Klisińska-Kopacz
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Autorzy pozostałych zdjęć w świetle dziennym: Pracownia Fotograficzna MNK
Źródło finansowania: badania wykonane w ramach działalności statutowej Muzeum Narodowego w Krakowie.

Conclusion

The carried out research allowed for determining the technology and technique of painting applied by Siemiradzki in his "Self-portrait". The results finally ruled out the possibility of it being the not yet found work painted for the Uffizi Gallery, though it is likely to be the only advanced study for the commissioned work. Nevertheless, the examined painting remains one of the best portraits by the artist, revealing the psychological traits and the temper of its painter.

Author of the concept: mgr Dominika Sarkowicz
Author of analytical research: dr Anna Klisińska-Kopacz
Author of analytical photographs: mgr Piotr Frączek
Authors of other photographs taken in daylight: Pracownia Fotograficzna MNK
Financed: research carried out within the statute activities of the National Museum in Krakow.

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[15] *Treasures of Poland. Rembrandt and the Precious Royal Collection*, katalog wystawy, Japonia 2010/2011.

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[1] W opracowaniach dotyczących twórczości artysty nie zetknęłam się z innymi autoportretami niż należące do krakowskiego muzeum: H. Siemiradzki „Autoportret" MNK II a – 1072 – po 1876 r., H. Siemiradzki „Autoportret z paletą" MNK II a – 1175 – ok. 1900 r.; oba niesygnowane.

[2] „Treasures of Poland. Rembrandt and the Precious Royal Collection", Japonia 2010/2011.

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[4] H. Blak, B. Małkiewicz, E. Wojtałowa, *Malarstwo polskie XIX w.* Katalog zbiorów pod redakcją Z. Gołubiew, Kraków 2001, s. 296.; informacja ta pojawia się również w innych pozycjach: M. Porębski, *Galeria Sztuki Polskiej XIX w. w Sukiennicach*, Kraków 2003; A. Król, *Henryk Siemiradzki (1843 – 1902)*, Stalowa Wola 2007.

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[7] J. Dużyk, *op. cit*., s. 339.

[8] *Op. cit*.; F. Stolot, *Henryk Siemiradzki*, Wydawnictwo Dolnośląskie, Wrocław 2001, s. 18.

[9] W odpowiedzi na kwerendę przeprowadzoną przez Muzeum Narodowe w Krakowie. Wątpliwość budzi jednak, być może mylnie odczytana, data tego zamówienia.

[10] Por. ze stratygrafią i opisem warstw technologicznych i chronologicznych poniżej.

[11] Zdjęcia w promieniowaniu analitycznym zostały wykonane w pracowni Lanboz MNK przez mgra P. Frączka.

[12] Wykryte pigmenty – patrz tabela 2.

[13] K. Pakuła, *Konserwacja szkicu pt.* Tancerka na linie *do obrazu* Linoskoczek *autorstwa Henryka Siemiradzkiego – przyczynek do rozpoznania problemu destrukcji dziewiętnastowiecznych malowideł na nietypowych podobraziach*, praca magisterska , Kraków 2005; s. 55. Badane szkice to: „Tancerka na linie" MNK II a – 1046, „Szkic krajobrazowy" MNK IIa – 1052, „Studium rąk" MNK II a – 1204, „Studium głowy mężczyzny" MNK II a – 1182, „Studium brodatego mężczyzny" MNK II a – 1188.

Streszczenie

„Autoportret" Henryka Siemiradzkiego, znajdujący się w zbiorach MNK, datowany na drugą połowę lat siedemdziesiątych XIX w., jest jednym z dwóch znanych, olejnych portretów/wizerunków własnych artysty. Według badaczy, jest to prawdopodobnie studium do autoportretu, zamówionego u Siemiradzkiego przez Galerię Uffizi we Florencji. Obraz nie jest ukończony. Zabiegi konserwatorskie przeprowadzone w 2010 r. stały się impulsem do dokładnego zbadania tak wyjątkowego obiektu. Na podstawie zdjęć w promieniowaniu analitycznym stwierdzono obecność jeszcze dwóch szkicowych kompozycji pod portretem mężczyzny. Analiza budowy technologicznej i techniki wykonania, połączone z identyfikacją składu warstw malarskich, zaprawy i podobrazia, pozwoliły określić warsztat malarski, z jakim mamy do czynienia w „Autoportrecie". Wnioski potwierdzają tezę, że może to być wyłącznie praca przygotowawcza do zamówionego dzieła. Rozpoznana budowa obrazu raczej wyklucza możliwość, aby była to jego ostateczna, lecz nieukończona wersja. Jeśli autoportret Siemiradzkiego dla znamienitej galerii rzeczywiście powstał, to,jak do tej pory nie został odnaleziony. Niemniej, badane studium pozostaje jedną z najlepszych prac portretowych artysty ukazując rys psychologiczny oraz temperament malarski autora.

Abstract

The "Self-portrait" by Henryk Siemiradzki, currently in the collection of the NMK, dated back to the second half of the 1870s, is one of the two known, oil self-portraits/likenesses of the artist. According to scientists, it is likely to be a study for a self-portrait, commissioned from Siemiradzki by the Uffizi Gallery in Florence. The painting has never been finished. Conservation treatment carried out in 2010, gave an impulse for a thorough examination of such a unique object. On the basis of photographs using analytical radiation, it was revealed that two more sketchy compositions existed beneath the portrait of the man. The analysis of its technological structure and technique of execution, combined with identification of the composition of layers of paint, ground and underpainting, allowed for determining the painting method we are dealing with in the "Self-portrait". Conclusions seem to confirm the thesis, that it might only be a preparatory study for the commissioned work. The identified structure of the painting seems to rule out the possibility that it could be its final but unfinished version. If the self-portrait of Siemiradzki for the eminent gallery had really been painted, then it has not been found yet. Nevertheless, the examined study remains one of the best portrait works of the artist revealing the psychological traits and the painting disposition of its author.

Sharaf El Din, Shahira[1]

Maladaptive use of a royal palace at Edfina, Egypt
Niewłaściwe wykorzystanie budynku królewskiego pałacu w Edfina, Egipt

Keywords: Royal palace, Edfina, maladaptive reuse, deterioration, historical value.

Słowa kluczowe: pałac królewski, Edfina, niewłaściwe wykorzystanie, destrukcja, wartość historyczna.

1. INTRODUCTION

Our built environment, like our outstanding and unique natural surroundings, provides a vital link to our past, assists in celebrating our achievements, and offers a vision for our future. It is a working, functional illustration of the many chapters in the story of our nation. Protecting our built heritage and preserving our national story for future generations presents a real challenge. A challenge that should be enthusiastically taken up by builders, developers, architects, community groups, heritage councils, individuals and all levels of government.

Egypt enjoys a great diversity of architectural heritage from the Ancient Egyptian to modern times. While Pharonic, Greco-Roman, Coptic, and Islamic monuments attract the attention of a wide range of international as well national scholars and conservation bodies, we find the "less fortunate" heritage – that is not listed – faces a lesser degree of attention from the general public [1].

Adaptive Reuse – Preserving our past, building our future highlights how our built heritage can be conserved through the successful combination of existing heritage structures and cutting edge architectural design. Therefore, the type of intervention on a heritage building depends on its cultural value, ranging from simple maintenance, where the objective is not to change the cultural value of the building, to complex rehabilitation, when it is intended to improve the performance of the building. It is commonly understood that adaptive re-use help extend the life of historic buildings and prevents them from becoming forsaken and derelict. It preserves building from changing outdated functions into new uses to meet contemporary demand.

Looking at the political impact on architecture after the 1952 revolution, one finds a lack of coordination between the different concerned authorities such as the Ministry of Culture, and the Ministry of Endowment. The ignorance on behalf of some of the Revolutionary politicians have lead to the nationalization and the seizing of some of the palaces and valuable buildings. These beautiful structures have inappropriately used as public elementary schools and police stations among other usages. However, In spite of the deterioration and misuse of these buildings, the rent control law and nationalization policy helped in preserving them against demolishing and rebuilding with better economic return [1].

2. HYPOTHESIS, PROBLEM DEFINITION AND STUDY METHODOLOGY

The monument subjected to study is in the city of Edfina at Beheira governorate. It is a huge wonderful palace from the Mohamed Ali royal era. Edfina palace was contributed to its significance as a historic district as it represented an important event which takes place in it (the summer palace of the royal family). However this palace which should be among the Presidential palaces or a touristic site, it was turned in to the faculty of Veterinarian in 1974, and that mal-adaptive reuse cause significant deterioration to the palace.

This study aims at evaluating the intervention with the heritage building through the following phases:

- The architectural and historical survey, which includes the acquisition of documented data about the building as follows: building location; building history; building construction stages; current usage; building style, aesthetic and visual qualities; the urban character and survey; movement and circulation systems; land uses and activities; and landscape documentation.
- Detailed survey of the existing condition of the building, which includes: degradation of building materials and elements; and deterioration factors.
- Elaboration of the diagnosis study (detailed survey).
- Criteria and suggestions for future intervention.

A detailed description of these phases will be presented on the following sections. Depending on the actual conditions of the building and on the objectives to be fulfilled, intervention can be assumed in different forms; from choosing an appropri-

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ate treatment plan for Edfina royal palace and its landscape till the restoration treatment for the historic palace through repair, alterations, and additions.

3. ARCHITECTURAL AND HISTORICAL SURVEY

The city of Edfina at Beheira governorate got a huge wonderful palace from the Mohamed Ali royal era. It is considered relevant to search to understand the historical development of the building so its transformation can be better deciphered.

3.1. Edfina Royal palace location

The coastal zone of Egypt on the Mediterranean extends from Rafah on the east to Sallum on the west for over 1200 km. It hosts a number of important residential and economic centers of the country including the cities of Alexandria, Port Said, Damietta, Rosetta and Matruh. Activities on the coastal zone include fishing, industrial activities, tourism, trading and agricultural activities in the delta region [2].

The region under study is in Edfina City which is located in EI – Beheira governorate (Muhafazat al Beheira), Egypt. Muhafazat al Beheira which is located about 135 km from the capital of Egypt – Cairo-From the East of Rosetta, and about 65 km from Alexandria. Beheira governorate enjoys an important strategical place, west of the Rosetta branch of the Nile. It comprises four important highways, namely the Cairo-Alexandria desert road, the Cairo agricultural road, the international road and the circular road. Beheira governorate is one of Egypt largest governorates and its capital is the "Damanhour" city. It is located in the east of the delta and surrounded from the north by the Mediterranean Sea while from the east there is the Rashid Branch from the west there are Alexandria and Matrouh governorates, and finally the Gizza governorate from the south. Edfina is a region of Beheira governorate.

Edfina got the huge wonderful palace from the Ottoman era which is the subject of this study. The palace lies on the western shore of the Nile River (Rosetta branch) north east of Edfina village at el Beheira governorate [3]. The old railway connecting the Montaza Royal palace and Edfina Royal palace is still working till today, as shown in Fig. 1. In spite of the transportation network from Alexandria to Rosetta is recently connected to the International Highway, still secondary roads are very narrow especially at some points of some settlements such as that near Lake Edku inlet.

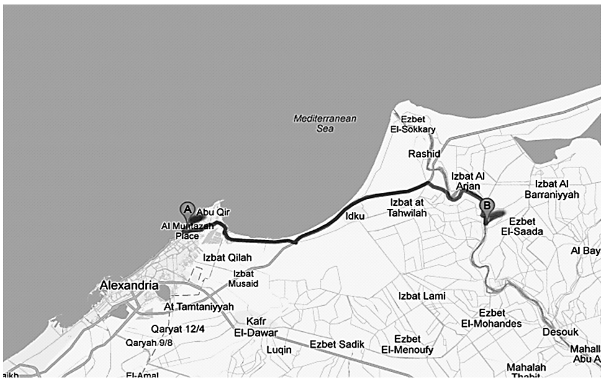


Fig. 1. Google earth map showing Al Montazah royal palace (A) and Edfina's palace (B).

3.2. History of the Royal Palace

It was under the government of Ismail Pasha (1863-1879) during the Ottoman rule, also known as Ismail the Magnificent, which the Europeanization of Alexandria began. Ismail built new roads and laid out new districts, improved trade relationships, and granted many plots of land, where numerous lavish palaces were built [4].

The royal palace at Edfina; shown in Figs. 2-3, was one of these palaces built in the 18th Century by the order of Khedive Ismail. Afterwards, King Fouad bought the palace from the government's surveillance on the former Khedive funds. The palace was surrounded by lands that were not reclaimed. Also the palace building composed only of ground and first floor. Then King Fouad the first decided to reclaim the lands using modern technical engineering tools and also to build new extension for the palace. In 1935, King Fouad ordered to build a floor over the old building and to build a new extension perpendicular on the old part. Also various agriculture building were built and a very well designed drainage and watering system was established .The palace became the destination of many kings and scientific committees. King Farouk continues his father works, adding a small part to the palace building. This palace was a gateway to king Farouk especially in the summer because of the weather in Edfina in Delta was chilling comparing to hot Cairo. King Farouk used to take the train from Montazah Palace at the eastern end till Edfina palace.

In 1961, Waguih Abaza turned the palace to become a governmental agricultural institute. In 1974 President Sadat decided to turn the palace in to the faculty of Veterinarian following the University of Alexandria according to presidential decree no.542.

However, as this palace should be among the Presidential palaces, in 2001the minister of Culture "Farouk Hosni" issued a ministerial decree No.233/2001 that is Palace and surrounded area should follow the high supreme council of antiquities and that the Faculty of Veterinarian should be transferred immediately to another building, but till now this order has not been implemented [3].

Fig. 2. The Edfina Royal Palace at 1979

Fig. 3. The Royal Palace at Edfina Nowadays

The palace have a marina directly on the Nile shore and extends along the eastern elevation, it's built of lime stone having the main entrance in the middle decorated with two big flower pots on both sides.. Plants shaped motives decorated the entire fence. The main entrance leads to eight half circular shaped steps from the middle & straight from the two sides. This marina used to be the anchorage in which the royal ships dock. This marina gave the palace a significant view all over the Nile.

It was from here that King Farouk used to take his yacht from Edfina palace marina till nearest resort passing through Edfina aqua ducts to reach his constructed resort in Rashid colony, enjoying the Nile and the Mediterranean at the same time, as illustrated in Fig. 4-5.

Fig. 4. Different views of the marina exposing the palace to the Nile

Fig. 5. The main entrance in the middle decorated with two big flower pots on both sides

3.3. The Royal palace Construction stages

The Royal palace was then built in three stages; as shown in Fig.6; the first stage was in the khedive's rule, while the second was in the king's Fouad the first and the third was in King Farouk era.

The first part of the palace was built in the ottoman period by the khedive Ismail who dedicated about 8000 acres for the palace and its surrounding gardens, this first part was only two floor height extending from north to south for 54 m long. The building faces the Nile and it was built of stone (el-fus) and the red bricks (masonry).

The second stage was built by king Fouad the first; he built a wing that extends from the east to the west perpendicular on the first building part. The main entrance of this wing is the western façade. This wing consists of three floors, the ground floor was specified for the kitchen, storages and the servants of the palace while the other two floors contains an inner lobby in which all rooms were related. This wing is characterized with its north balconies which contains granite columns and distinguished capitals. The letter (F) was set on motives on the façade referring to the king who constructed this part, as shown in Fig. 7. The third stage was an extension done by king Farouk, it was a small part attached to the original palace (the khedive's wing) from the south east direction, as illustrated in Fig. 6 [5].

3.4. The Royal palace status and usage

The Edfina Royal palace is used till now as The Faculty of Veterinary Medicine. The faculty was founded in 1974. Study started in the academic year 1975/1976 with a capacity of 50 students. The faculty provides veterinary medical services for the purpose of fostering the animal resources and increasing the awareness among the members of the community. Several centers are affiliated with, the faculty, providing poultry and animal treatment services to individuals, and institutes.

Degrees Offered in the faculty are: Bachelor of Veterinary Medical Sciences, diploma, Master of Veterinary Medical Sciences and Doctor of philosophy in Veterinary Medical Sciences. The faculty encompasses about 17 departments, such as: Anatomy & Embryology, Cytology & Histology, Physiology, Pharmacology, Biochemistry, Animal husbandry, Food Hygiene, Animal Nutrition, Pathology, Microbiology, Poultry & Fish Diseases, Theriogenology, Surgery, Animal Medicine, Forensic Medicine & Toxicology, Hygiene & Zoonotic Diseases and Parasitology [6].

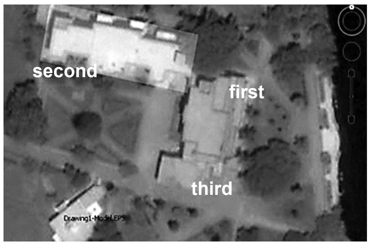


Fig. 6. The construction stages

Fig. 7. The letter (F) was set on motives on the façade refers to the king who constructed this part

3.5. Land uses and activities

The palace is surrounded by the following buildings: Anatomy department building, lecture hall building, a small clinic, new mosque, old mosque, an under construction hospital, girl students' dorms and a garden pergola; as shown in Fig. 8.

3.6. Landscape documentation

The garden of Edfina Royal Palace contains very rare plants from all over the World, as shown in Fig. 9. Some of these plants have survived till now surrounding the palace from all sides. Also an old octagonal shape marble fountain decorated the palace landscape from the North facing the north elevation of kings Fouad extension.

3.7. Building style, aesthetic and visual qualities

The palace architecture features vary from the Romanesque windows to the angled bay window, as shown in Fig.10. The palace buildings encompass new gothic revival architecture represented in the turret with battlements and pinnacles in the western and eastern elevation. The tower has square shaped plan, it was built from the red bricks with south direction entrance and having windows all around its four facades.

The northern and southern facades of the old palace were romantically asymmetry with tall narrow windows, Fig.11. Italianate architecture features also can be revealed in loggias and balconies with wrought-iron railings, or Renaissance balustrade, Low-pitched or flat roofs and projecting eaves supported by corbels. Motifs drawn from the Italianate style were incorporated into the building vocabulary, Fig. 10. Distinguish belvederes faced the Nile and the marina.

A black & white paved gravel mosaic insets are embraced with the whole palace building from the southern and western direction, as shown in Fig. 12. The entrance of the palace is decorated with two gravel mosaic tiles with large motifs to form unique carpets. By assuming & redrawing of the existing carpets, the designs in Fig. 13 imitate their original state.

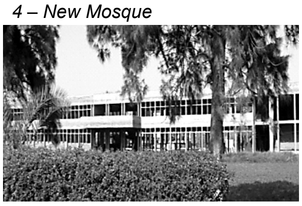
1 – Anatomy department building

2 – Lectures' building

3 – Clinic

4 – New Mosque

5 – Old mosque



6 – Hospital under construction

7 – Girls students dorms

8 – Garden Pergola

Fig. 8. Buildings surrounding the Royal palace and land uses

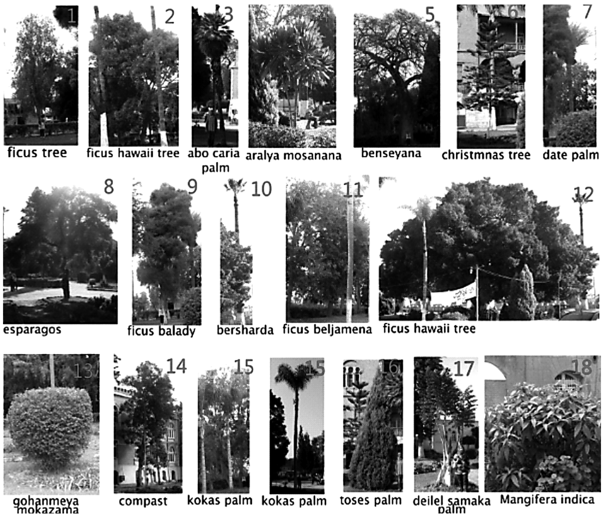


Fig. 9. Documentation of the garden of Edfina Royal Palace with its rare plants

Turret with battlements and pinnacles *Tall windows* *Angled bay window* *Battlements and cornice*

Fig. 10. The palace architecture features

Fig. 11. Northern and southern facades of the old palace

In King Fouad part of the palace, some pharaonic stones with hieroglyphic writing appeared in the eastern elevation with a very unique feature, Fig 14. Also very rare column capital in which the architect used a feminine sphinx capital appeared in the same façade, as illustrated in Fig. 15. A motif with the letter "F" referring to the king who constructed this part remains there.

The palace interior is a mix between the Italianate and Oriental styles. In Italianate interior decoration there were direct parallels to "Italianate" architecture with free recombination of decorative features drawn from Italian 16th-century architecture and objects, which were applied to purely 19th-century forms. Crown moldings, chair rails, Dentils egg and darts are common architectural details, Fig. 16. Linoleum, ceramic and wood are the most common flooring options. Some oriental features appear in using wooden crafts which were rare in the Italianate style. Also the combination between the Italianate and oriental style is obvious in the iron of handrails of the spiral stair with remarkable stained glass used in windows as shown in Fig. 17.

Fig. 12. Two gravel mosaic tiles with large motifs to form unique carpets at the palace entrance

Fig. 13. Redrawing the two gravel mosaic carpets to imitate the original design

Fig. 14. Pharaonic stones with hieroglyphic writing

Fig. 15. Very rare column capital in which the architect used a feminine sphinx capital

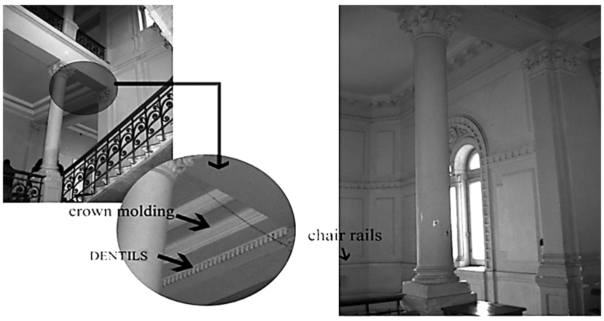


Fig. 16. Crown moldings and dentils egg in the ceiling, and chair rails as common architecture feature

Fig. 17. Stair with remarkable stained glass used in windows, and its current status

4. DETAILED SURVEY OF THE PALACE'S EXISTING CONDITION

4.1. Degradation of building materials and elements

Precise documentation has to address all available information about a monument and all types and results of investigation and work, ranging from the identification of a stone object to the quality control of preservation measures. Today, documentation plays an important role in education and project funding in the field of monument preservation. Modern computer technology represents an important tool for manifold steps of documentation.

The monument mapping method has been established as a non-destructive procedure for the precise registration, evaluation and documentation of lithotypes and deterioration phenomena [7]. The documentation is to ensure the thorough and proper understanding of all therapeutically actions ranging from their decision up to their long-term control and the maintenance of monuments.

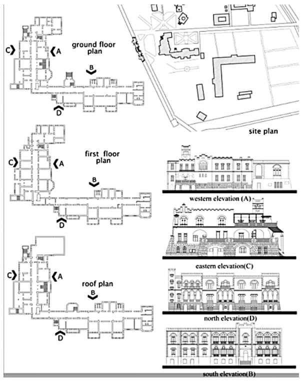


Fig. 18. Royal Palace main facades drawings

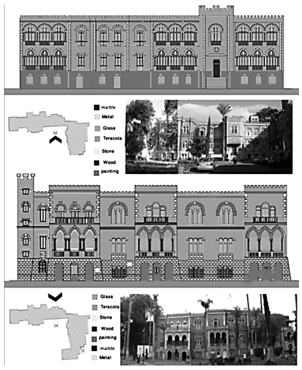


Fig. 19. Monument mapping of the south and north elevation of the palace respectively (litho types).

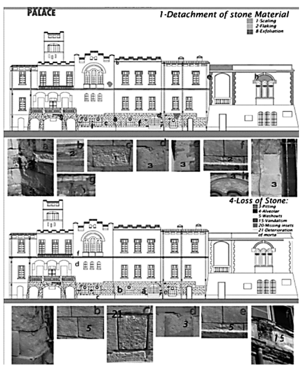


Fig. 20. Monument mapping details in the west elevation (weathering features)

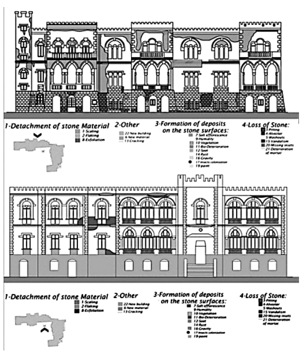


Fig. 21. Monument mapping of the north and south elevation (weathering features)

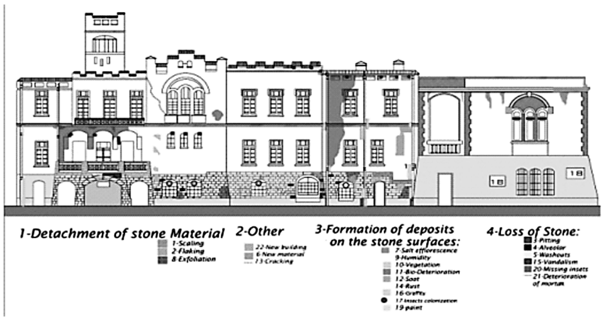


Fig. 22. Monument mapping of the west elevation (weathering features)

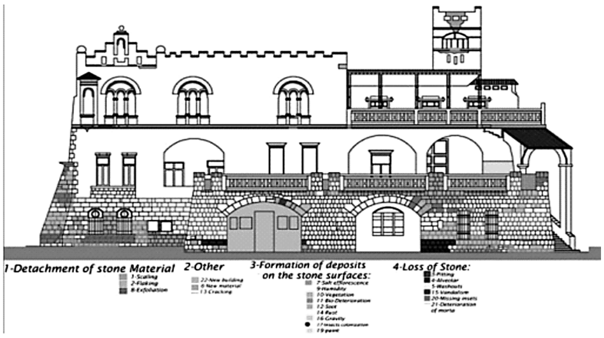


Fig. 23. Monument mapping of the east elevation (weathering features)

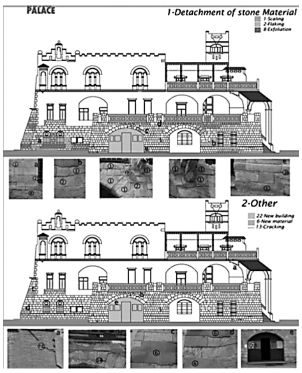


Fig. 24. Monument mapping details in the east elevation (weathering features)

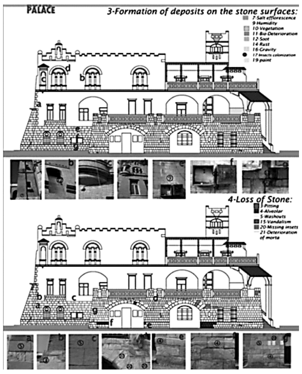


Fig. 25. Monument mapping details in the east elevation (weathering features)

Diagnosis of degradation of building materials and different type of damage to building elements are accomplished through visual examination and documentation of different litho types and deterioration mapping of the four facades, as documented in Figs. 18-25.

Throughout the study and analysis we found out the major factors or problems affecting this historical palace and its surroundings can be divided into two main groups: internal factors and external factors.

4.2. Deterioration factors

4.2.1. Internal factors

Foundation depth; water accumulation which is obvious in the main entrance mosaic settlement and some external cracks are signs for structural changes in the wall, as the wall settles with time and can lead to building failure. Shallow foundations also must be maintained.

Human intervention; many of the palace rooms were turned into laboratories without any isolation work. Changing usage without any consideration of loads, materials durability and the adjacent spaces can destroy the palace; each wall panel stands on itself and is stabilized by a floor or roof slab. Thus, if there is any slight disturbance of the wall due to other external factors, the wall panel cracks or even collapses. Also ceramics tiles replaced marble in many spaces due to financial restrictions leading to more leakage problems.

Mortar composition; some of the fine aggregate used does not seem to have a proper grading as required. This is due to the fact that, they were mined from any possible sources in the neighborhood of the construction site. It is well known that the choice of a well graded aggregate improves the strength of the mortars.

Building enclosure; the roof, gutters and downspouts and walls protect the building's interior from moisture penetration that causes deterioration of the building materials. Openings in the roof and walls, such as ventilation pipes, windows, and doors, must be appropriately constructed and well maintained to keep water out of the structural system and building interior. The roof drainage system (gutters, downspouts, and discharge system) is frequently a cause of water penetration and deterioration. The roof drainage system must be maintained to direct water away from the building.

4.2.2. External factors

Lack of maintenance; another cause of structural failure in the historical buildings is lack of maintenance. Lack of maintenance begins a sequence of deterioration that typically involves the breakdown. For example, roof leakages have contributed to serious damage to the buildings structural systems. Also, some of the mangrove poles supporting floor and roof slabs have rotten, sagged or cracked, which resulted to slabs collapsing.

Vegetation; Moisture and humidity resulted in covering some facades with vegetations. Big trees have deep roots that destabilize building structures. Also, these trees pose a threat to the building structure in the event that one or both fall.

Bio-deterioration; Lichen and algae affect the wall of the historical buildings. The activity of both organisms does not result in major changes, but the appearance of the walls is altered, Fig. 26. In addition, they have a corrosive effect on the substrate because they release acid metabolites [8].

Lack of knowledge; Original building materials are generally renewed without understanding their material characteristics and identification of their deterioration problems. Modern building materials are frequently used without testing their compatibility with the original materials. For example, the wide employment of cement during restoration has created irreversible damage to the historical masonry. Indeed, cement-based mortar restrains movement and leads to stress that cause failure in the original masonry, as shown in Fig 26. Moreover, some of the new restoration materials are not as durable as required while others do not permit the transfer of internal moisture out, Fig. 26. Modification of existing openings and change of openings design is also a crucial human intervention that falsifies building history. Signs fixations can also lead to external wall deterioration.

Fig. 26 Photos showing different deterioration aspects such as human intervention, back weathering, bio-deterioration and water penetration respectively.

5. Elaboration of the diagnosis study

The palace must be restored to match its design during the king's Farouk era and also all surrounded historic features will respect the palace architecture style. There will be two stages; active stage which will contain Cleaning and restoration procedures and adaptive re-use for the palace suggestions;

and passive stage which will discuss preservation of the site through different steps.

5.1. Active plan

5.1.1. Cleaning and restoration methods:

A well-planned cleaning project is an essential step in restoring rehabilitating, or preserving, a historic masonry building. Proper cleaning methods and coating treatments, when determined necessary for the preservation of the masonry, can enhance the aesthetic character as well as the structural stability of a historic building. Removing years of accumulated dirt, pollutant crusts, stains, graffiti or paint, if done with appropriate caution, can extend the life and durability of this historic resource. Suggested treatment for the four palace facades are cited in Table 1.

Table 1 Suggested treatment for the four palace facades

External Facade	Main deterioration problems	Treatment suggestions
The eastern elevation	detachment of stone materials (scaling flaking and exfoliation)	Dentistry and consolidation
The north elevation	Formation of deposits on the stone surfaces (salt deterioration and vegetations).	Poultice desalination techniques and Alkaline cleaner's method [8].
The western elevation	Formation of deposits on the stone surfaces (soot and rust), and also some cracks as signs for wall settlement.	Water or water mists, washing cleaner method. Fixing cracks, consolidation for foundation [8].
The south elevation	formation of deposits on the stone surfaces (paint, bad conservation and salt efflorescence).	Chemical method by using alkaline cleaners' method.

Sometimes treatment for the complete deteriorated materials (wood, stained glass, and wooden handrail is needed. If the treatment failed then replacing by new one identical in shape, color, texture, materials, and scales in order to preserve the historical image come as second choice.

As for stones affected by bio-deterioration, it is preferred to be clean regularly, either by chemical or water cleaners. The stone affected by salt efflorescence we can use consolidate which reduces the salt efflorescent and protected the stone from being further deteriorated (scaling, flaking, exfoliation, alveolar, and pitting).

For Terracotta Bricks affected by bio-deterioration are preferred to be clean only regularly, either by chemical or water cleaners. The Bricks affected by salt efflorescence, we can use water repellent which reduces the salt efflorescent and protected the stone from being further deteriorated.

5.1.2. Adaptive re-use for the palace

The adaptive reuse of a historic building should have minimal impact on the heritage significance of the building and its setting. Main interventions with the palace buildings are summarized in Table 2. Developers should gain an understanding of why the building has heritage status, and then pursue development that is sympathetic to the building to give it a new purpose. Adaptive reuse is self-defeating if it fails to protect the building's heritage values.

Table 2 Summary of the main interventions with the palace building

Alterations observed in the palace building		
Causes	**Needs**	**Interventions**
Building usage	Separate circulation	Division of spaces.
	Service spaces	Removal of connecting elements.
	Extra spaces	Use of spaces for different functions. Mass /door addition.
Structure problems	Repair or replacement	Replacement of surface materials to cheaper and less durable material, Fig. 19. Replacement of architecture elements.
Changes in style	Semi-open spaces	Window blocking on ground floor, Fig. 19.
	New service spaces	Mass addition.
	Mechanical equipment	Installation of new equipment.

5.2. Passive plans

Preservation of the site can be accomplished through associated steps. Urban analysis for the site revealed that the palace has two main axes, as given in Fig. 27. The main axe is from the main entrance gate leading to the shore and secondary axe from secondary gate facing the green house from one side to mosque and El-makam in the opposite side. These two axes can divide the site into two main areas, as shown in Fig. 28:

Restrict area with restricted intervention which means only to restore the palace and re-use it as touristic palace and open museum. The reusing of the palace as faculty of Art may be acceptable as the students will be the palace restorer. This will begin by removing the veterinary college to another place and to restore all the feature of the site beginning from the small detail inside the palace up to the outer landscape. Restrict area consists of main royal palace, the main Marina, the green house the mosque and the El-Makam . The rare landscape composed of old trees and plants can turn into a botanical open museum and labeling each tree with full description including age. Unwanted buildings such as buildings for classes, a cafeteria, an animal barn and small shops can be removed.

Unrestricted area in which all new built buildings can be removed or altered freely such as the hospital under construction, student dorm, a restaurant and parking.

Revitalization of Edfina palace is not only about conservation and preservation of the heritage building, but also about some aspects that need to be put on focus such as social, economic, culture, people activities and environmental facilities. The improvement of the Edfina Area will make it livable and attractive place for people to come These are many actions that can encourage this trend such as:

Marketing Zone creation as an open space which can create attractive activities and comfortable for people to stay. This can encourage the number of visitors to come to this area.

Converting unoccupied buildings to become commercial area, such as shopping centre, hotel, café, royal restaurant, bookshops and public offices.

Royal Boat, marina and Waterfront Area usage and enhancement. Developing the public facility along the Nile side area can encourage the study area to become attractive and livable. The creation of new activities can be done by developing

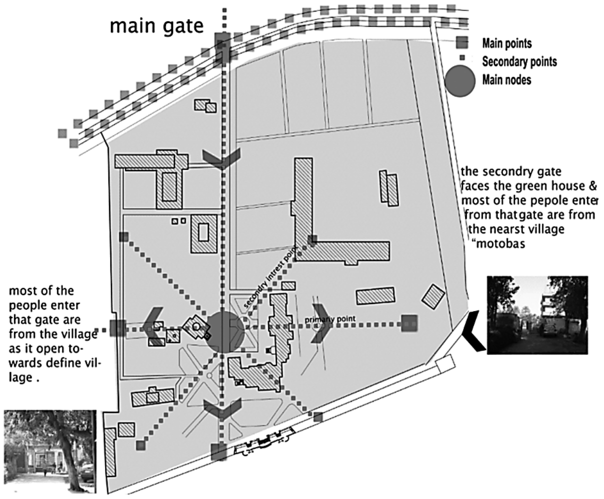


Fig. 27 Site two main axes

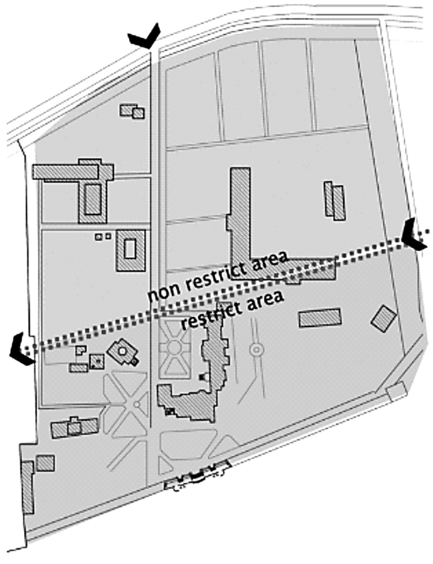


Fig. 28 Dividing the site into restricted and unrestricted areas

market, outdoor café and shop lot, which can attract people to come and enjoy stay in while fishing and sailing around.

Royal Maze and gardens and the improvement of accessibility and Facilities. Creating obvious guided easy access of people to this area with appropriate signs and maps. Facilities such as pedestrian walk, street and public transportation are also needed. Also, separation between street and pedestrian walk make comfortable and safety travel for people in the study area. The redevelopment need to focus on the comfortable and safety of disable people facility.

Recreation zone and public places formation. Creating public place is considered essential to the function of Edfina Palace revitalization plan to turn historic area to tourism place; it must be a main focus in its development. Public place is not just an open space, also the place to facilitate the activity of visitors, it must be an attractive place to support people to come and enjoy staying in. Public place also can provide various activities in this area such as bookshops and restaurants.

Create new activities in the study area relating to the social and economic assets. Activities such as Horse Riding Zone and botanic museum in the rare garden around the palace, can attract many people to come to this area especially tourists to visit the museum, look around, taking photographs and go to their office.

6. CONCLUSION AND RECOMMENDATIONS FOR FUTURE INTERVENTION

Creating new activities can improve and enhance Edfina palace development plan. This will be considered in the concept and its function as a public place, which will provide the various recreational and entertainment, and have various activities such as educational, art, culture, urban heritage and creative community. Finally, the design guideline can be a recommendation to do revitalization of Edfina palace. The recognition and understanding of attraction activity and public facility which need to be provided is the main focus in the redevelopment and revitalization of a historic area to be a public and tourism place. Standards for rehabilitations of the whole siteare as follows:

Study and understanding the people activities and economic sector are important in redevelopment and revitalization of the historic area, which will improve the area to become attractive and livable.

The revitalization should point on physical and non-physical element, because both of elements have interaction in term of depth study of historic area.

The study and analysis of the public facilities enhancement is needed to facilitate the activities in the area, particularly to redevelop the study area.

A property will be used as it was historically, or be given a new use that maximizes the retention of distinctive materials, features, spaces, and spatial relationships. Where a treatment and use have not been identified, a property will be protected and, if necessary, stabilized until additional work may be undertaken. Regular maintenance and monitoring for risk reduction of heritage properties must be accomplished.

The historic character of a property will be retained and preserved. The replacement of intact or repairable historic materials or alteration of features, spaces, and spatial relationships that characterize a property will be avoided.

Each property will be recognized as a physical record of its time, place, and use. Work needed to stabilize, consolidate, and conserve existing historic materials and features will be physically and visually compatible, identifiable upon close inspection, and properly documented for future research.

Changes to a property that have acquired historic significance in their own right will be retained and preserved.

Distinctive materials, features, finishes, and construction techniques or examples of craftsmanship that characterize a property will be preserved.

The existing condition of historic features will be evaluated to determine the appropriate level of intervention needed. Where the severity of deterioration requires repair or limited replacement of a distinctive feature, the new material will match the old in composition, design, color, and texture.

Chemical or physical treatments, if appropriate, will be undertaken using the gentlest means possible. Treatments that cause damage to historic materials will not be used.

Design guidelines for the building restoration and the site will be according to the Egyptian Urban Harmony guides [9].

The adaptive reuse of a historic building should have minimal impact on the heritage significance of the building and its setting. Also maintenance is central to protecting cultural significance or value because it is the least destructive of all the 'interventions' which inevitably occur in the process of conserving historic buildings. Adaptive reuse of buildings has a major role to play in the sustainable development of communities. When adaptive reuse involves historic buildings, environmental benefits are more significant, as these buildings offer so much to the landscape, identity and amenity of the communities they belong to. Our lifestyle is enhanced not just from the retention of heritage buildings, but from their adaptation into accessible and useable places. The reuse of heritage buildings in established well reused areas can provide the community with new opportunities. The adaptation of heritage buildings presents a genuine challenge to architects and designers to find innovative solutions that retain heritage significance.

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Abstract

The city of Edfina at Beheira Governorate got a magnificent Palace from the Mohamed Ali Royal Era called The Royal palace Edfina or king Farouk's Palace. That palace, 20 km from Rosetta, was built in the 18th Century by the order of Khedive Ismail. Then King Fouad I decided to build new extension to it. It was built according to the Italianate style and it was king Farouk resort especially in summer because of the fine weather in Edfina in Delta as compared to hot Cairo.

In 1961,Waguih Abaza, first Beheira governor, turned the palace to become the headquarter of some governmental Agricultural society. In 1974, President Sadat decided to turn it in to the faculty of Veterinarian following the University of Alexandria according to presidential decree no.542. Already this palace should be among the Presidential palaces and thus the minister of Culture Farouk Hosni in 2001 issued an order No.233 that this Palace should follow the high supreme council of antiquities and that the Faculty of Veterinarian should be transferred immediately to another building. Till now this order has not been implemented.

This paper presents the Royal palace historical and artistic value. It also illustrates the maladaptive reuse and the deterioration processes of the palace. Regenerating this building can reinforce a sense of community, make an important contribution to the local economy and act as a catalyst for improvements to the wider area, i.e. Beheira governorate. New uses of this palace should be allowed with sensitive oriented adaptations.

Streszczenie

Miasto Edfina w prowincji gubernatorskiej Al-Buhajra szczyci się wspaniałym pałacem z okresu panowania Mohameda Ali, zwanym królewskim pałacem w Edfina lub pałacem króla Farouka. Pałac ten, znajdujący się 20 km od Rosetty, został zbudowany w XVIII wieku z rozkazu Kedywa Ismaila. Następnie król Fouad I postanowił go rozbudować. Został wzniesiony w stylu włoskim i stał się ulubionym letnim kurortem króla Farouka ze względu na przyjemny klimat w Edfina leżącej w Delcie, w porównaniu z upalnym Kairem.

W 1961 roku, Waguih Abaza, pierwszy gubernator Al-Buhajra, oddał pałac na potrzeby rządowego towarzystwa rolniczego by służył jako jego główna siedziba. W roku 1974, Prezydent Sadat zdecydował się przekształcić go na Wydział Weterynaryjny Uniwersytetu w Aleksandrii, dekretem prezydenckim nr 542. Ponieważ pałac powinien znajdować się na liście pałaców prezydenckich, Minister Kultury Farouk Hosni w 2001 wydał zarządzenie nr 233 by podlegał najwyższej radzie starożytności, a Wydział Weterynaryjny miał zostać natychmiast przeniesiony do innego budynku. Jak do tej pory to zalecenie nie zostało wprowadzone w życie.

Ten artykuł przedstawia historyczne i artystyczne wartości królewskiego pałacu. Ma także zilustrować niewłaściwe wykorzystanie i zachodzące w nim procesy destrukcji. Rewitalizacja tego budynku może wzmocnić poczucie wspólnoty, przyczynić się znacząco do rozwoju lokalnej gospodarki, a także zadziałać jak katalizator dla pozytywnych zmian na większym obszarze, to znaczy w prowincji Al-Buhajra. Nowe funkcje pałacu powinny być wprowadzane poprzez wyważone adaptacje.

L. Van Parys[1], J. Noël[2], D. Lamblin[3], S. Datoussaid[4], A. Tilmant[5]

Disorders in north transept of the cathedral in Tournai (Be): Structural monitoring, simulations and temporary stabilization

Uszkodzenia konstrukcji w północnym transepcie katedry w Tournai (Be): Monitorowanie konstrukcji, symulacje i tymczasowa stabilizacja

Keywords: Heritage, Cathedral, Tournai, Masonry, Monitoring, Simulation, Stabilization

Słowa kluczowe: dziedzictwo, katedra, Tournai, konstrukcja murarska, monitoring, symulacja, stabilizacja

1. INTRODUCTION

A great part of important heritage buildings have suffered to be forgotten for several centuries. As the public authorities have now clearly understood the necessity to transmit these master pieces to the next generations, the structural experts and their mastery of computational techniques become unavoidable in restoration works. This paper deals with the Our Lady cathedral in Tournai (BE), part of UNESCO World Heritage, unfortunately suffering of soil problems having induced structural disorders affecting its nave, its choir as well as its transept, the latter being the focus of the present paper.

2. HISTORICAL & ARCHITECTURAL PRESENTATION OF THE CATHEDRAL

Two thousand years ago, around 75 km from the North Sea shores, Roman people decide to settle down on the top of a hill, not far from the Escaut riverside. Some centuries later, the place is still busy. Religious habits have evolved; two churches have been erected on the former Roman site and later led to destruction (fire and/or invaders). During the 12th century, it is decided to build a cathedral on their ruins. The articulation between the 12th and the 13th century combines the end of erection of the Romanesque nave, transept and choir as well as the arrival of a new bishop deeply interested by northern practices associated with the Gothic architecture.

In this framework and intending to install an immense cathedral for marking his power, he initiates the construction of a new choir, erecting a huge apse several meters behind the Romanesque choir that is gradually dismounted as the Gothic one progresses closer and closer to the transept zone. Once the Gothic choir has been finished, a probable lack of money made the erection of a Gothic nave and Gothic transept in accordance with the first plans impossible, providing this way the impressive contrast between a slender and widely opened choir and a massive and robust nave and transept that remained nearly unchanged until today.

The unexpected source of architectural richness radiating from this combination of both the most important European medieval architectural styles from a single building has encouraged UNESCO to recognize the Our Lady cathedral as part of the World Heritage.

3. STRUCTURAL DISORDERS IN THE TRANSEPT OF THE CATHEDRAL

Further than these stylistic patterns those have clearly inspired the architecture of other great cathedrals in northern Europe, the visitor of Our Lady often surprised by the great number of signs denoting the existence of significant structural disorders. Such signs may be detected on pavements, walls, columns as well as vaults. The **crack patterns** constitute a first family of signs. They essentially affect the northern part of the transept where various structural masonry members are impacted: the three-ring diaphragm arch and the barrels vaults it supports, the floors and vaults

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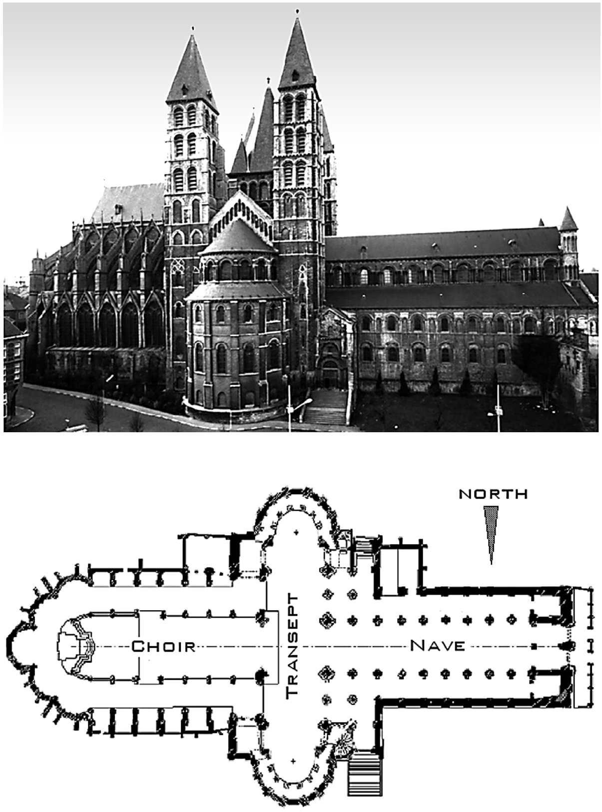

Fig. 1. North elevation of Our Lady cathedral (left) – plan view (right)

of the northern apse path as well as the walls of the major stairway located inside the Brunin bell tower are clearly concerned. Areas that are not accessible to the visitors definitely confirm the trend: wide opened cracks affect the impressive


Fig. 2. Crack in wall upside vaults (left) – pillar settlement (centre) – global inclination (right)

masonry walls supporting the roof system upside the vaults. The **differential settlements** represent a second family of signs. They affect portions of walls as well as particular pillars with sensible amplitude. They can easily be detected by respectively considering horizontally built stone bands (some of them are clearly inclined) or the vertical position of sculpted pillar basements relatively to the stone floor level that has been replaced some times during the 12th and 13th century and is then rather flat and horizontal (some of them are almost 10 cm embedded in the pavement). The **global system deformations** constitute a third family of signs. Hardly noticed through a local focus, they definitely appear as soon as a concerned system may be considered globally. Walking from the railway station, the severe inclination of the Brunin bell tower towards the west (80 cm at the top of the tower) visually appears in proximity with the four other bell towers. Through these signs, the cathedral clearly expresses the existence of pathologies implying its structure.

4. INTERDISCIPLINARY DOCUMENTATION OF PATHOLOGIES CONTEXT

Wide opened cracks, differential settlements and noticeable deformations led to outline as a priority to dispose of a sharp survey of the altered morphology. Therefore, a topographical survey has been initiated that allowed to objectively setup the knowledge of the current morphology: not the entire Romanesque structure appears to be submitted to settlements. In fact, a fictitious line inclined by almost 45 degrees on the conventional north-south direction should be considered for dividing the Romanesque part [1, 2]. Both the western thirds are affected by a settlement that keeps growing as we move away from the fictitious line while the eastern third of the Romanesque structure did never move. The architectural organization of the building (symmetry in shapes and roofs, unity of masonry material and similar window pattern) being not likely to provide any intrinsic heterogeneity in the structural response, the potential impact of an underground problem could clearly be supposed. In this framework, information concerning both the foundation system and the soil had to be collected.

As historians announced that no written data was available for documenting the Romanesque foundation system, several localized excavations have been realized in strategically chosen parts of the building (northern and western side of the nave, along the inclined Brunin bell tower).

These works were carefully carried out by a team of archaeologists under the direction of Professor BRULET (Université Catholique de Louvain – BE). They revealed precious information [3] bringing to light some historical elements of the birth and life of the building. The remains of the previous constructions were partly unearth and the works allowed recognizing the morphology, pattern and levels of the actual foundation system that is composed of impressive and continuous stone masonry walls (external leaves and mixed filling), homogeneously dispatched under external walls and internal pillar rows. In parallel to this information, the manual excavations also allowed confirming the clear continuity of each severe crack affecting the upper parts of the building throughout the foundation, leading to the total disconnection of elements in some places. In this framework, the continuous foundation system designed by the medieval builder for ensuring the stability of the Romanesque structure now appears to act as a set of independent blocks, namely in the problematic zone surrounding the Brunin bell tower. Continuing the manual digging action down to the basis of the foundations made it possible to observe the nature of the material on which the foundation is posed: soft soil in some places or carbonate bedrock in others.

The discovered heterogeneous nature of soil under the foundation justified the interest for a geotechnical investigation campaign. Carried out namely under the direction of Professor TSHIBANGU (Université de Mons – BE), it privileged the recourse to on-site drilling followed by lab analysis of collected cores. Achieving a structured testing grid was not easy partly due to the difficulty of entering testing devices through existing bays inside the cathedral (internal grid) and partly due to the deep embedding of the cathedral inside the urban tissue (external grid). Nevertheless, the obtained information advantageously completed with some geophysical measure-

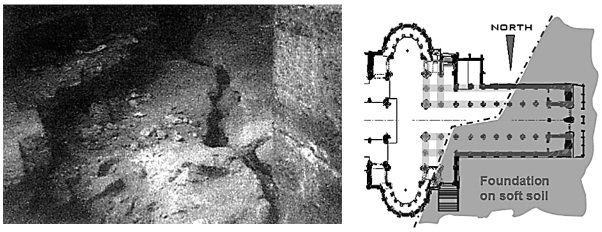

Fig. 3. Crack in foundation walls under pavement (left) – synthesis of collected information (right)

ments allowed establishing a 3D model of the underground stratification in the cathedral area: the layer of soil interposed between the top of the carbonate bedrock and the bottom of the foundation system reveals thicknesses varying from 0 up to 6 meters, sometimes abruptly (about 5 meters of vertical delta on less than 3 meters of horizontal delta) and the quality of this soil appears to be globally poor.

5. SIMPLIFIED FE MODELS FOR VALIDATING ASSUMPTIONS

Stone carrier used for ancient constructions (explaining the occurrence of steps in the level), karstic problems usual in the concerned region [4] (explaining the top bedrock alteration and poor quality of soils), modified water flows associated to seepage in local extractive activities (explaining the importance of settlements)… Although many interrogations remain concerning the soft soil layer, it clearly appears that its intrinsic nature should probably be considered as cause for explaining the observed disorders although establishing the validity of such an assumption is not easy. Implied in several European preservation committees, Professor BARTHELEMY (Université de Mons – BE), gathered Professor MACCHI (Pavia – IT) who carried out FE calculations on the Pisa tower and Professor HALLEUX (Université Libre de Bruxelles – BE) who managed the calculations in the restoration of Brussels Town hall tower. Together, they outlined modelling guidelines aiming at achieving a partial or total confirmation of an eventual relationship between the soft soil state and a consequent occurrence of various structural disorders affecting the Romanesque nave and transept. The elaboration of a FE model has been initiated, in this spirit, by a team directed by Professor LAMBLIN (Université de Mons – BE) [5]. The proposed study focuses on the problematic north transept and its neighbouring nave parts although southern parts are also involved in the model. The sharp morphology of the model has been established from the combination of a topographical survey and archaeological investigations. The physical and mechanical values that are required for the models are derived from published data in complement to laboratory tests performed on samples cored from masonry walls or stone pillars.

The masonry material is modelled using a macro-modelling philosophy: a fictitious homogeneous material is considered whose equivalent properties are computed on the basis of both material and morphological aspects. This fictive material is assumed to exhibit an isotropic, linear and elastic behaviour. Although no limitation is introduced concerning the compressive solicitations, a Rankine model is considered concerning the tensile solicitations: it is implemented under the shape of a discrete cracking process taking place in an iterative framework. On the basis of the general stress state associated to the iteration # i, the geometry of the mesh is adapted in order to introduce a discrete crack by node duplication at the point associated to the higher value of maximal principal stress and orthogonally to the related principal direction if this higher value is above the tensile strength for the considered masonry. Such an approach has already been applied with success [6] and is recognized to outline valid stress repartitions although the strain range is often underestimated.

The interaction between the studied masonry system and its direct neighbourhood is smartly managed. The effect of non-represented building parts (namely the junction with the choir) is taken into account through constraints applied along interfaces where no absolute motion has occurred since the erection. The effect of stone vault is replicated through equivalent point loading whose values are computed in an external FE calculation. The tie bars have been effectively modelled with truss bars as they are likely to play an active role in load redistribution. The effect of the roof system is taken into account through equivalent line loading whose values are estimated in a classical manner. The mesh is structured and relies on an 8-node-brick approach. Although the effect of wind loads has been proposed for some particular studies, the main calculations are carried out under the essential action of gravity. The developed models gave quite interesting qualitative results. So the influence of the disorders can be noticed and parametric studies can be carried out to highlight the relative variation of the stresses and strains but the absolute value given by the software cannot be considered in itself due the assumptions introduced in the model (see before).

The soft soil layers interposed between the top of bedrock and the bottom of the masonry foundations has been carefully modelled as it definitely drives load redistribution in our statically redundant structure. In practice, a preliminary approach

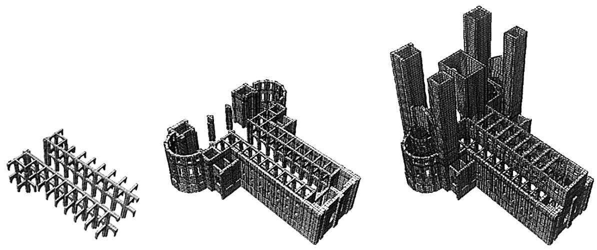

Fig. 4. Erection of the simplified FE model for the transept and nave system

with Winkler approximations[1] appeared to be insufficient as it was not likely to effectively capture the pressure bulb development and its potential effects. Then, the effective presence of soil material has been privileged, a simplified constitutive model being chosen as only the impact of deformations on the masonry system was interesting to be replicated. The bottom face of the soft soil is constrained for simulating the effect of the subsequent bedrock. Lateral earth pressure acting on each underground masonry foundation walls have been computed in the Lower Limit State according and applied as equivalent surface loads on the model.

A modelling of the first steps of the construction of the cathedral revealed that already shortly after their building, the foundations would have been submitted to excessive traction in the area of the good/poor quality soils transition line [7]. In this way, the superstructure of the cathedral seems to have been built on weakened foundations. Following these results, the assumption that the foundation system suffered from cracks in their early life could explain the fact that pathologies started to affect the newly built cathedral only decades after its erection as attested by historical documents.

At the scale of the entire model, the crack management allows to express interesting conclusions. Although a tensile stress limitation through a smeared cracking method could give suitable primary results, the recourse to a discrete approach where cracks are introduced as geometrical discontinuities enhances the quality of the models, outlining results that are closer to the real situation. In particular, the location and morphology of numerical crack patterns, the calculated settlement trends as well as the global deformations predicted by the proposed FE models appear quite in adequacy to the reality.

A routine developed by J. NOËL and improved by J. COUVREUR likely to automatically manage the evolution of cracking process in the models through nodes duplications [7] allowed to precisely follow the crack propagation inside the global system. It clearly appeared that the more the cracks progresses in the gable wall, the arch and the apse, the more the bell tower get inclines itself [8, 9]. The proposed simplified FE approach gives good clues concerning the causes for most of the pathologies concerning the north gable wall, the north apse of the transept and the inclination of the Brunin bell tower. Indeed, the presence of an important gradient in soil quality under the Brunin bell tower could explain some of the pathologies highlighted in the models and visible *in situ*. Three corners of the Brunin bell tower are posed on a thick layer of bad quality soft soil although the last one is directly posed on the bedrock. The massive stone structure of the bell tower makes an eventual settlement of soil layers to be significant.

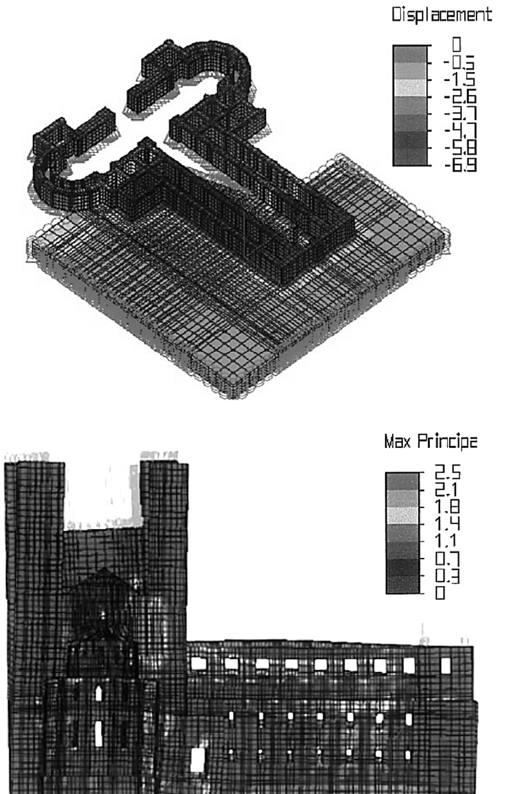

Fig. 5. Settlement for foundation and soil model (left) – tensile stressed zones in the system (right)

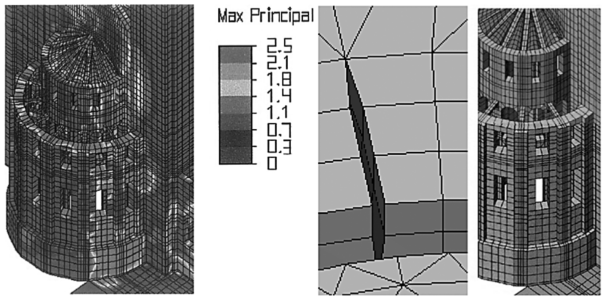

Fig. 6. Tensile stresses around Brunin bell tower (left) – node duplication & concerned zones (right)

The differential effect induced by the heterogeneous posing conditions brings the occurrence of parasite solicitation inside the system (bending, tension, shearing) that induce the development of disconnections, opening the way to still important inclination motions.

6. EVOLVING CHARACTER: SUPPOSITIONS AND RISK

Specific hydrological considerations concerning potential natural underground water flows between upper and lower area of the city (towards Escaut river) present the occurrence of soil particle transportation as a possibility, removing soil and potentially enforcing the evolving character of measured and modelled effects. Combined with the morphological configuration of disconnected blocks inside the Brunin bell tower and based on geometrical considerations, it was shown that the damaged system has got the possibility to avoid the convergence towards a stable equilibrium state. Some topographical measurements carried out on a limited period tended to confirm the effectiveness of evolution trends. Therefore, the recourse to a temporary stabilization structure preventing any increase in damages has been studied. Nevertheless, with such a structure likely to strongly constraint any further motion, it is not possible to easily detect evolutions any more as its installation "comes and breaks the thermometer". In this framework, a monitoring system coupled with the temporary stabilization structure and likely to manage a permanent structural survey has been proposed. It should detect local motions (with limited amplitude due to the constraints) and also provides information concerning the evolution of forces inside the structure (with great value due to constraints).

Designing an urgency stabilization structure for a cathedral submitted to significant soil motions is not usual and then not particularly easy. To become installed on a complex site, it has to be ingenious. The proposed solution relies on post-stressed steel bars for temporarily anchoring the Brunin bell tower inside undamaged bells tower along both the east-west and the north-south direction. For making the system to be efficient, the convenient localization of bars should be achieved as well as the choice of a suitable section and repartition system for avoiding potential failure occurring in the steel system or in the still undamaged masonry parts. The effective design and the related impact study has been carried out with FE models derived from the one presented previously. A 10mm settlement increase under the Brunin tower has been introduced in the cracked model and the efficiency of steel bars has been analysed. It gave birth to the solution that has been effectively: 2 bars × 50 mm bars placed at level 25,5 m from pavement with 6500 daN post-stressing each in the east-west direction, 4 bars × 50 mm bars placed at level 18,5 m combined with 4 bars × 50 mm bars placed at level 26,5 m with 5000 daN post-stressing each in the east-west direction. According to the FE studies, the chosen configuration of stabilization (position, number and post-stressing force) is likely to reduce the crack openings induced in gabble walls by any further settlement for more than 60%.

The prescription of the coupled permanent auscultation system was carried out under the direction of Professor BOLLE (Université de Liège – BE). It is composed of several complementary captors that are usually used for monitoring the global behaviour of civil engineering equipment like dams or tunnels. A review of some of them is proposed hereafter: it outlines the usual requests associated to each of them, the principle of the measurement and the classical usage in civil engineering.

An automatic pendulum has been installed on a pillar that leans against the Brunin bell tower. This device aims to measure the relative movement of two vertical points. The wire bearing the pendulum is anchored at the upper point and the reading device measure automatically the variation of the inclination of the wire. The pendulum is immerged in water in order to damp the movements and avoid vibrations and oscillations. The pendulum gives a measure of the global inclination of the pillar. Some clinometers are also recording the local variation of the tip angle of some critical structural elements of the cathedral. The variation in the inclination is measured through a small gravitational pendulum coupled

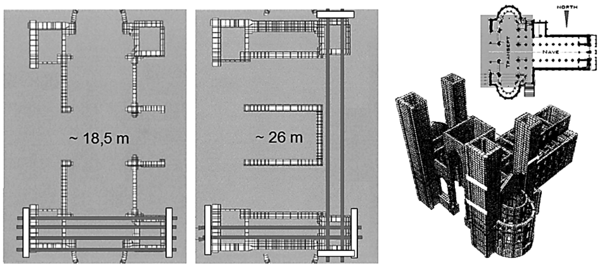

Fig. 7. urgency stabilization system with post-stressed steel bars designed with FE models

with electronic measuring equipment. Crack opening captors are monitoring the evolution of the cracks size in the transept, the apses, the arches and the tower. Vertical movements of the pillars of the transept and choir are monitored using soil settlement gages consisting of several tanks connected to each other by a liquid saturated tube. In each reservoir, a floating mass is sensitive to the variation of liquid level due to settlements. One reservoir is chosen as the reference level and the device gives then the relative vertical displacements of all the elements on which a tank is fixed.

7. EVOLVING CHARACTER: CONFIRMATIONS AND CONSEQUENCES

The day-to-day monitoring analysis was carried out by the team of Professor LAMBLIN, focussing on the data treatment and producing deliverables that could be interpreted by Professor BOLLE. The total interpretation of data collected along several years will not be discussed in the present paper. Nevertheless, it is important to notice that further than the

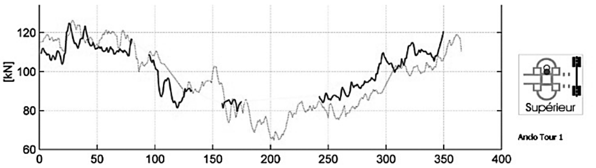

Fig. 8. Evolution of forces in upper bars along the east-west direction (on 365 days)

seasonal effects associated with meteorological conditions, the system clearly shows an evolving character for the phenomena: the complex masonry system is not yet arrived to a stable position, confirming the feared evolution trends.

In such an evolving case, the simplified FE models presented here have shown that the problem of Brunin bell tower stability was far to be the most critical aspect: the tower is still able to submit an important inclination before collapsing under its self-weight. Nevertheless, a three-ring arch supported by the Brunin bell tower is submitted to springing displacement and this is recognized to be problematic although the literature proposes only poor information concerning such problems. The complete study of this particular problem will not be detailed in the present paper but is proposed elsewhere [10].

CONCLUSION

Since the experts have been asked to study the Our Lady Cathedral issue in 1965 [11], the building has been under permanent attention. The growing pathologies it suffers needed trans-disciplinary studies to be carried out. Engineers gathered architects, archaeologists and historians to find solutions to preserve this historical construction. These lasts focused on the birth, life and evolution of the building. Their excavations have put into light the hidden face of the building, its foundations and the previous constructions they are settled on. Using this information and after *in situ* and in lab investigations, engineers have analysed the stability of the edifice. They could highlight some of the causes of the observed pathologies and their work led to the installation of monitoring systems and temporary strengthening devices which are guaranteeing the survival of this exceptional part of the Belgian and World Heritage. The use of all the technological knowledge and the sensibility of all the present actors have managed to protect the Cathedral structure without taking away the aesthetic and cultural interest of the prestigious edifice. The evolving characters of the phenomena justify the setup of a permanent stabilization solution where important challenges will have to be taken up.

ACKNOWLEDGEMENTS

The authors would like to thank P. THIJS and C. BURY from the Technical Department of Buildings at the Province de Hainaut. Together with ir A. TILMANT, they succeeded in gathering specialists from various disciplines around the questions posed in one single project, making a period of threats for the cathedral to become an impressive chance to make a better future.

[1] Classical soil-structure approach where springs are connected at each node of the bottom face of the foundation with an individual rigidity computation based on soil properties as well as local layer thickness

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Abstract

A great part of important buildings involved in the World Heritage have suffered to be forgotten during several centuries. As the public authorities have now clearly noticed the touristic potentialities and definitely understood the necessity to transmit these master pieces to the next generations, structural experts have often to play a great role in restoration works.

This paper concerns the Our Lady cathedral in Tournai (BE) whose architectural richness has encouraged UNESCO to recognize it as part of the World Heritage at the beginning of the 21st century. Unfortunately, serious geotechnical problems have, along the centuries, induced the occurrence of structural disorders that affect now the roman nave and the gothic choir as well as the "transition style" transept that constitutes the subject of the present paper.

After a quick introduction to the concerned historical and architectural contexts, the paper presents most of the pathologies that may be directly captured by direct observation in the transept of the cathedral. Then, the interdisciplinary campaigns carried out for achieving a better knowledge of the situation are described; the collected information is summarized as well as the Finite Element studies that have been carried out in order to get a sharp understanding of the problems. They are based on simplified macro-models taking a crack propagation process into account and they outline the interest to install a monitoring system proposing a permanent structural survey.

The effectively installed system coupling pendulum, settlement gages, crack opening captors and clinometers is described. The interpretation of the information collected during a first period is proposed and the interest to design a temporary stabilization structure is then discussed. The simplified Finite Element simulations leading to the effective design of these structures are described and the solution that has been effectively installed is illustrated, briefly summarizing the challenges associated with the implementation of a future permanent stabilization of affected masonry arch systems.

Streszczenie

Duża grupa ważnych zabytkowych budynków z Listy Światowego Dziedzictwa trwała w zapomnieniu przez kilka wieków. Jednak obecnie władze wyraźnie dostrzegły ich turystyczny potencjał i zrozumiały konieczność zachowania tych arcydzieł dla następnych pokoleń, toteż eksperci specjalizujący się w tematyce konstrukcji zaczęli odgrywać istotną rolę podczas rewaloryzacji tych obiektów.

Niniejszy artykuł dotyczy Katedry Notre-Dame w Tournai (BE), której architektoniczne bogactwo skłoniło UNESCO do uznania jej za część Dziedzictwa Narodowego na początku XXI wieku. Niestety, w ciągu wieków poważne problemy geotechniczne wywołały uszkodzenia konstrukcji, występujące w romańskiej nawie i gotyckim chórze, jak również w transepcie w „stylu przejściowym", który stanowi przedmiot poniższej pracy.

Po krótkim wprowadzeniu w kontekst historyczny i architektoniczny obiektu, artykuł przedstawia najpoważniejsze nieprawidłowości jakie można dostrzec w trakcie bezpośredniej obserwacji transeptu katedry. Następnie opisano analizy interdyscyplinarne przeprowadzone dla uzyskania lepszego rozeznania w sytuacji; podsumowano zebrane informacje, podobnie jak badania metodą elementów skończonych wykonane w celu dogłębnego zrozumienia problemu. Bazują one na uproszczonych makro-modelach uwzględniających proces propagacji pęknięć oraz proponują zainstalowanie systemu monitorującego pozwalającego na stałą obserwację konstrukcji.

Opisany został skutecznie zainstalowany system, składający się z wahadła, mierników osiadania, czujników wychwytujących pęknięcia oraz inklinometrów. Zaproponowano interpretację informacji zebranych podczas pierwszego okresu obserwacji, a następnie przedyskutowano kwestię czasowej stabilizacji konstrukcji. Opisano uproszczone symulacje metodą elementów skończonych, prowadzące do optymalnego projektowania takich konstrukcji, oraz rozwiązanie, które zostało skutecznie wprowadzone, krótko podsumowując wyzwania związane z wdrożeniem planowanych rozwiązań trwale stabilizujących naruszone systemy kamiennych łuków.

Alessandro Baratta[1], Ottavia Corbi[2]

The static behavior of historical vaults and cupolas

Zachowanie statyczne historycznych sklepień i kopuł

Keywords: Masonry behaviour, Masonry vaults, Structural assessment, No-Tension model

Słowa kluczowe: statyka elementów murowanych, murowane sklepienia, ocena konstrukcyjna, model beznaprężeniowy

1. INTRODUCTION

Masonry is the main material mankind has exploited to provide itself a shelter.

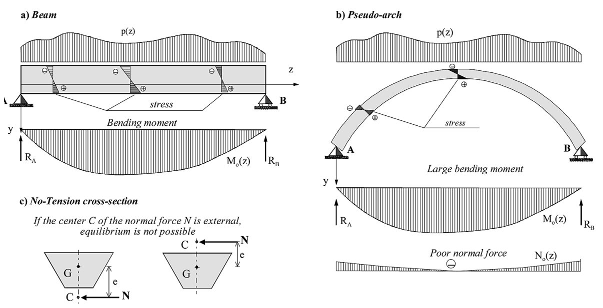

Fig. 1. a) Beam requires compression/tension stresses to be resisted; b Curvature of the beam does not help by itself to cancel tension; c) If the eccentricity e is large No-Tension equilibrium cannot subsist

Homes, temples, offices, markets and so on are built by some kind of masonry since the beginning of civilization. Walls are the main way loads are transferred to foundations and to underlying soil, but horizontal floor structures require some more skill, since masonry, due to its very poor, unreliable, inhomogeneous and time-degrading, tensile strength, is not able to resist bending moments. This is the reason why masonry buildings are often complemented by wood or, more recently, steel systems to cover spaces, providing beam elements resisting by pure flexure.

In a simply supported beam, equilibrium is sustained by bending moments, which in turn require that compression stresses are coupled with tension (Fig. 1a); so if some beam has ever been attempted it is soon realized that failure is inexorable.

On the other side, early ante-literam architects learned from nature that it is possible to overpass empty spaces by stones: natural arches are encountered everywhere in the world (Fig. 2).

a) b)

Fig. 2. Natural Arches: a) Capri (Italy); b) The "Elephant Arch" in Pantelleria (Italy)

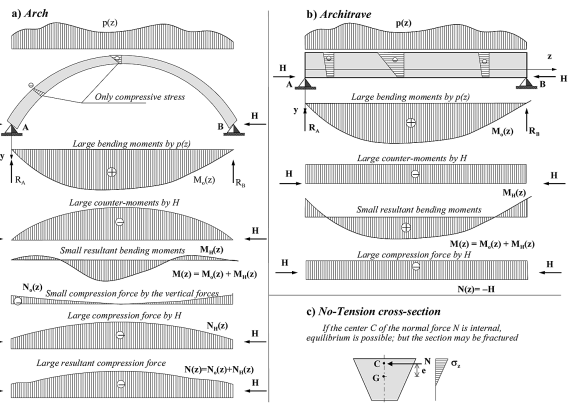

Fig. 3. a) True arch: the action of the thrust force H increases the normal force and mitigates the bending moment. A small eccentricity e = M/N results; b) The same happens for a beam with horizontally contrasted suports, thus resulting in an architrave; c) The eccentricity is small and the center C of the normal force is in the interior of the cross-sections

So, the first character that is acquired at glance is that the masonry should be "curve". If one consider a curved beam (the *pseudo-arch* in Fig. 1b) one finds that the only difference is the insurgence of a compressive normal force on the cross-section, which mitigates, but does not cancel, the need for tension (Fig. 1b). The bending moments remain the same, the normal force is small, the eccentricity is large and the center of the force is

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generally out of the cross section (Fig. 1c): equilibrium cannot subsist unless tension is resisted where it is necessary in the structure. If the system is horizontally fixed at both ends, a new entity is born, the *horizontal thrust H*, which drastically changes the static regime and a true *arch* is realized. The thrust force is the key for the arch statics. It acts, in fact, producing larger compressive normal forces and strong counter-moments (Fig. 3a), thus mitigating the flexure and enhancing compression. The effect is quite independent on apparent curvature: the important fact is that a horizontal force exists able to produce counter-moments with respecto to active load. The architrave is nothing else than an arch with the appearance of a beam (Fig. 3b). The typical condition is a composite compression-flexure stress, where the compression is large and the flexure is small, so that the eccentricity with respect to the central line is strongly reduced and the center of force enters in the interior of the cross section: equilibrium is now possible by purely compressive stresses (Fig. 3c).

So, Man learned that it was possible to cover spaces with stones. Anyway, he found and inhabited also large caves, so the attempt to reproduce nature (a strong impulse in Architecture, as testified also in recent times by the Gaudì's opera, see e.g. [1]) may be has pushed to realize double-curvature roofs. This activity gradually resulted in a success, with larger and larger spans being covered, thus leading to the early architecture and to its developments up to our times.

The historical development of Structural mechanics is exhaustively reconstructed in the book by E. Benvenuto [2]. A very interesting and complete historical survey on the conception, realization and progress in the masonry vaults technology can be found in [3] and in [4, 5]. Here an observation by Thomas Young is reported, namely: "*The construction of the dome is less difficult than that of an arch since the tendency of each arch to fall is counteracted not only by the pressure of the parts above and below but also by the resistance of those which are situated on each side*…..". That double curvature surfaces are easier to be built than simple arches or barrel vaults is a fact that will receive further specification in the sequel.

2. THE MASONRY AS A MATERIAL

2.1. Overall properties

Masonry is not properly a "material" in the strict sense of the word. It consists in the (generally man-made) assemblage of a basic component (the stones) simply laid on each other or, more often, jointed by mortar. Stones and mortar may have very variable mechanical properties, and the way in which the stones are organized in the masonry volume may (the masonry "texture") may be very different, and is subject to the skill and the creativity of the designer and/or of the builder.

So, "masonry" has not a uniquely defined object, and it is very difficult to set up a mechanical model able to closely reproduce the properties of masonry, fitting all the possible variety of masonry assortment and texture.

Anyway, in all structural analyses the engineer is forced to balance the trend to reproduce the material (and consequently the structural behaviour) as closely as possible, with the practical manageability of the analytical tools. Linear theory of structures applied to steel, reinforced concrete and even to masonry, is a successful example of such effort. In all cases the basic theory should include the major features of the behaviour, possibly neglecting many details that poorly influence structural safety assessment, and/or are uncontrollable. The small tensile strength in concrete, for instance, not only yields a poor contribution to the structure performance, but since it is a highly uncertain parameter in the concrete mass of a building, it increases uncertainty of the analysis' results: so it is preferred to adapt linear theory by neglecting tensile strength rather than to exploit cumbersome procedures yielding results depending on uncontrollable parameters.

The first step is then to identify the major properties, that are more or less common to all masonry types. The basic knowledge can be achieved through simple experiments. Uni-axial compression/tension tests can be performed on some Representative Volume Element (RVE) of a typical masonry (Fig. 4).

After some experiments, it is possible to conclude that (Fig. 4a): i) the masonry has different elastic moduli in tension (E_t) and compression (E_c); ii) the masonry has different limit stresses in tension (σ_t) and compression (σ_c); iii) the limit stress in tension is much smaller than the limit strength in compression ($\sigma_t << \sigma_c$); iv) the behaviour at failure in compression has some degree of ductility; v) the behaviour at failure in tension is definitely brittle, so tensile strength cannot be recovered absolutely.

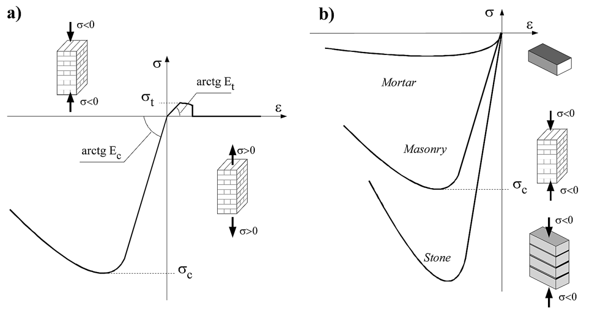


Fig. 4. a) A typical test of compression/tension on a masonry speciimen; b) The limit strength in compression is intermediate between the strength of mortar (small) and the strength of the bricks (large)

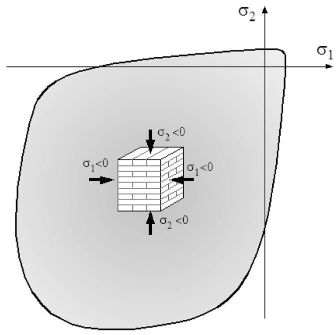


Fig. 5. Synthesis of biaxial tests on masonry prisms. Limit domain (see e.g. Hegemier [6] and Page [7]

Moreover, surprisingly (Fig. 4b), the limit strength in compression of masonry is larger than the strength of the weak element (the mortar) and is bounded from above by the limit strength of the strong component (the stones); this is due to some complex phenomenon of stress interaction and transverse deformation of mortar with respect to stones. It is also easy to understand that if the axis of the stress is rotated by an angle, say 90°, the results of the experiment may significantly change, in particular as regards the tensile strength. Some similar conclusions can be drawn from biaxial tests (see e.g.

[6, 7]). Experimental limit strength domains are of the type in Fig. 5, showing a high capacity in compression and a very poor limit in tension without ductility.

Summing up, masonry is a non-linear material, strongly hetero-resistant, anisotropic with respect to tensile strength, with compliance coefficients depending on the orientation of the stress axes and different in compression and tension, and with brittle failure at the tension threshold.

2.2. Influence of the texture on masonry properties. The case of the masonry wall

The influence of the texture on the masonry performance can be illustrated by the following example. Assume that a panel is built by regular bricks with interposed poor mortar joints, lacking any adhesive power. Consider that bricks are set according to the following two patterns (Fig. 2.3a and 2.3b)

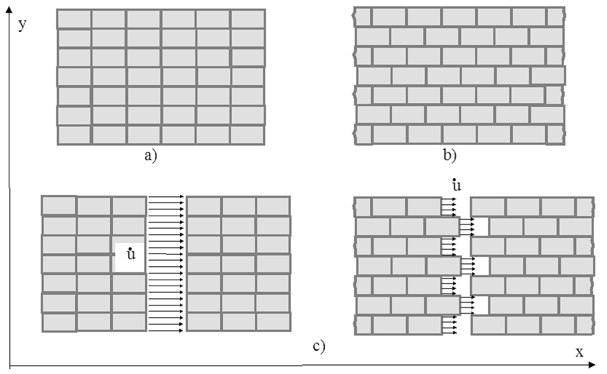


Fig. 6. Masonry element: a) Aligned bricks; b) Staggered bricks; c) Free lateral expansion for both panels

If there is no vertical compression both panels are free to expand laterally without encountering any resistance (Fig. 6c).

If a vertical compression is applied, the panel in Fig. 6a still can freely separate; by contrast an horizontal tensile pseudo-strength becomes active in the panel in Fig. 6b, because of friction and interlocking of bricks with each other. The failure mechanism in Fig. 7b can be studied for the bi-dimensional masonry plane element in Fig. 7a having a friction coefficient *f*, a joints stagger *s* (Fig. 7c) and a row density ω defined as the ratio between the number of block rows in the panel height *H* and the height *H*. In Fig. 7 $\omega = 7/H$.

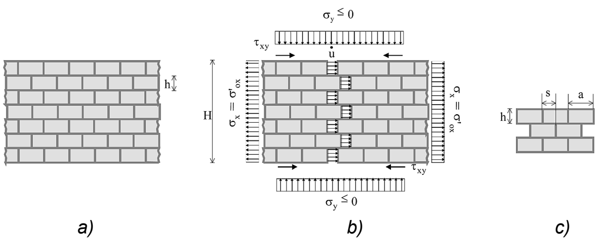


Fig. 7. Masonry element: a) Geometrical dimensions, b) Failure mechanism under compression and limit tensile forces, c) Stagger parameter

The wall is subjected to vertical compression stresses σ_y orthogonal to the joints direction and horizontal tractions σ_x parallel to the joints. It is possible to prove [8] that the horizontal tensile strength σ'_{ox} is given by (Fig. 7b)

$$\sigma'_{ox} = -f\sigma_y s\omega \tag{1}$$

The ratio between the compressive stress on the joints and the transverse tensile strength is

$$\left|\frac{\sigma'_{ox}}{\sigma_y}\right| = fs\omega \tag{2}$$

If the length of the stone is *a*, *s* is of the order *a*/2. Usually $a > 2h$ (very often $a > 4h$), with *h* the thickness of the brick, and so $s > h$. On the other side, $\omega \approx 1/h$, so that $s\omega > 1$ (very often $s\omega > 2$). With the help of mortar and/or of roughness of the interface between stones, *f* may possibly be rather large ($f = 0.5 \div 0.8$), and the ratio in Eq. (2) is frequently larger than 1, i.e. the tensile strength in the direction parallel to joints is larger than the acting compressive stress.

It can be also proved that a pretty ductility is associated to the tensile strength σ'_{ox}. With reference to the diagrams in Fig. 8, applying a safety coefficient γ to the limit resistance σ'_o, the loss in strength is balanced by a gain in ductility. In other words if σ'_a is the admissible stress and δ_a is the maximum ductility, one can write

$$\sigma'_a = \sigma'_o/\gamma \quad ; \quad \delta_a = \frac{\varepsilon'_{oa}}{\varepsilon'_a} = 1 + \frac{\varepsilon'_r}{\varepsilon'_o}(\gamma - 1) \tag{3}$$

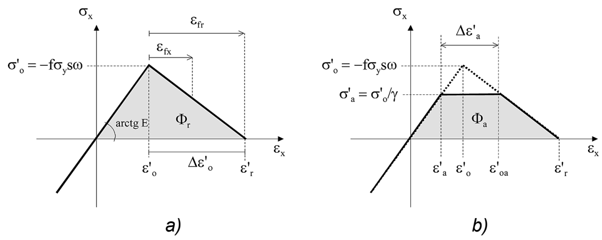


Fig. 8. a) Stress vs. deformation in the tension range, b) conventional diagram with variable ductility

A fundamental observation is that Eq. (1) not only expresses the tensile resistance of the masonry element, but also puts to evidence that the tension can be contrasted in function of the static needs by means of a skilled orientation of the texture of the masonry blocks and of the mortar joints. After recognizing that by the combined effect of compression and friction the lines of the mortar joints are probably the lines where original designers and builders intended to provide tensile strength in the masonry mass, it can be conceived that a technical practice had spread out, very similar to the modern technology of reinforced concrete where the structural designer inserts steel bars in way to balance tension along stretched lines.

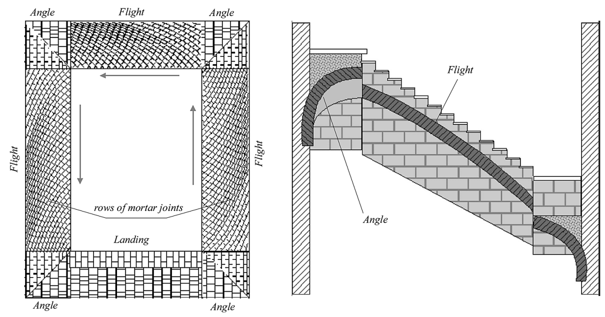
Many examples proving that clever architects were aware of this effect when designing vault structures can be illustrated.

3. MASONRY TEXTURE AND APPARATUS IN VAULT STATICS

3.1. The cantilever stairs

In the static analysis of a vaulted staircase, like in Fig. 9, it is possible to recognize three basic typological components: the landings, the angle connections, and the flights of stairs (two or three depending on the structure morphology). The structure is supported by the outside walls system which represents the stairs box.

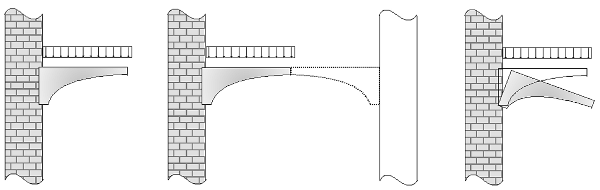
Looking at the section of a vaulted stair in Fig. 10a, such structural conformation suggests an apparent paradox: despite the fact that masonry is not effective in sustaining tension stresses and bending, it should work as a cantilever, or however it is an incomplete vault which lacks the counter-thrust from the missing part of the arch (Fig. 10b) and so being prone to lose the equilibrium state (Fig. 10c).



a) b)

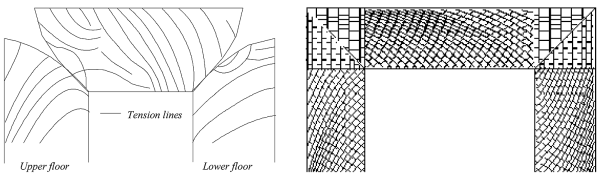
Fig. 9. Vaulted stairs: a) Planimetric view; b) Longitudinal section

It is quite obvious that the solution of the contradiction goes pursued abandoning the search of improbable plane patterns and by investigating three-dimensional equilibrium paths accounting for the space articulation of such structural organisms, searching stress fields in equilibrium and compatible with the resistant abilities of the masonry material as usually interwoven in the case of "cantilever" stairs.



a) b) c)

Fig. 10. Transverse sections of vaulted stairs: a) Section and particular of one step; b) "Half barrel vault" model, c) Improbable "cantilever" behaviour



a) b)

Fig. 11. Comparison of tension isostatic lines with the mortar rows. a) Tension lines calculated by a FEM (linear) procedure; b) Mortar rows in the flights

After identifying the basic internal force distributions through which the stairs can equilibrate their own weight and live loads, and the correlation that was intended by the original builders between statics and masonry tissue, it is also possible to design the reinforcement of the vaults, that shall be designed in way to sustain the possible equilibrium paths. Apart from complex analyses (see e.g. [9]), it is possible to identify simplified equilibrium patterns that are compatible with the load-carrying capacity of the structure [10]. All these approaches, FEM and/or simple 3D-beam, agree in identifying isostatic tension lines that approximately agree with the proceeding of the rows of mortar joints (Fig. 11), that are compressed in the orthogonal direction, thus developing a tension capacity along their lines of action; thus proving that the statics of these stairs are strictly connected with the vault apparatus. It is also possible to use this argument in an inverse fashion, i.e. to infer isostatic lines proceeding from the observation of the masonry texture. Any double curvature cover, in fact, is a highly hyperstatic system, which means that it can select its own pattern in a large set of possible equilibrium paths. So texture and vault apparatus are a tool by which, apart from the shape of the vault (barrel vault, rib vault, groin vault, etc.) the architect can steer the structure to work in some preferred way.

3.2. Tension in spherical domes

Consider the axial-symmetric hemispherical dome with radius R and thickness t (Fig. 12a), supporting its own weight w, where it is well known that in the classical solution, tension should be active along the parallel lines after some degree of the zenith angle $\varphi = 51.8°$.

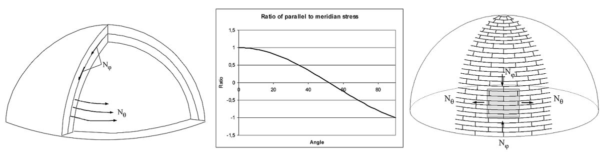
Here the meridian stress N_φ and the hoop stress N_θ are [11]

$$\begin{cases} N_\varphi = -\dfrac{wR}{1+\cos\varphi} \\ N_\theta = wR\left(\dfrac{1}{1+\cos\varphi} - \cos\varphi\right) \end{cases} \tag{4}$$

with $w = \gamma t$ and γ the unit weight of the material constituting the shell.

The ratio is

$$\frac{N_\theta}{N_\varphi} = 1 - \cos\varphi - \cos^2\varphi \tag{5}$$

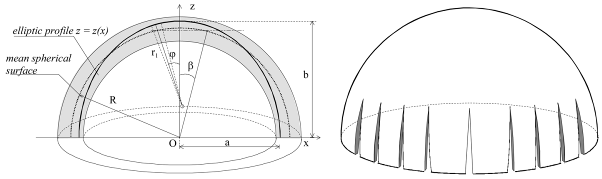


a) b) c)

Fig. 12. a) Spherical dome; b)Ratio of parallel to meridian stress resultant. Nj is everywhere compressive for any j, and Nq is a tensile stress for j > 51.8°; c) The friction pattern yields a admissible stress if fsw > 1

The ratio is plotted in Fig. 12b, whence one can see that the ratio is always not larger than 1. So, if masonry is organized by staggered regular bricks –as often happens– tension could generally be faced by the friction mechanism as illustrated in Sec. 2.2 (Fig. 12c).

Anyway, equilibrium can be found by some other membrane surface other than the mean surface of the shell, provided it is included in the thickness between the (spherical) intrados and extrados. Considering a revolution membrane surface having an elliptic profile with radii a and b, included in the interior of the hemisphere (Fig. 13a) the internal forces equilibrating the weight of the spherical dome can be found by the following procedure



a) b)

Fig. 13. a) The elliptic membrane surface included in the dome thickness; b) Possible physiological fractures in the masonry

Consider the spherical cap above the center angle β, whose weight is

$$W = 2\pi w R^2 (1-\cos\beta) \tag{6}$$

The angle β is related to the zenith angle φ by the relationships

$$\sin\beta = \frac{a\sin\varphi}{\sqrt{a^2\sin^2\varphi + b^2\cos^2\varphi}} \;;\; \cos\beta = \frac{b\cos\varphi}{\sqrt{a^2\sin^2\varphi + b^2\cos^2\varphi}}$$

$$d\beta = \frac{ab}{a^2\sin^2\varphi + b^2\cos^2\varphi}d\varphi \tag{7}$$

The radii of curvature of the ellipsoidal surface are ([11], p. 40)

$$r_1(\varphi) = \frac{a^2b^2}{\left(a^2\sin^2\varphi + b^2\sin^2\varphi\right)^{3/2}} \;;$$

$$r_2(\varphi) = \frac{a^2}{\left(a^2\sin^2\varphi + b^2\sin^2\varphi\right)^{1/2}} \tag{8}$$

so that

$$\sin\beta = \frac{ar_2(\varphi)\sin\varphi}{a^2} = \frac{r(\varphi)}{a} \;;\; \cos\beta = \frac{b}{a}\frac{r(\varphi)}{a\tan\varphi} \tag{9}$$

$$with\;\; r(\varphi) = r_2(\varphi)\sin\varphi$$

and

$$d\beta = \frac{a}{b}\frac{r_1(\varphi)}{r_2(\varphi)}d\varphi \tag{10}$$

The equilibrium versus the vertical translation can be written

$$2\pi r_2 N_\varphi(\varphi)\sin^2\varphi + W = 0 \tag{11}$$

and

$$N_\varphi(\varphi) = -\frac{wR^2(1-\cos\beta)}{r_2(\varphi)\sin^2\varphi} \tag{12}$$

The ellipsoidal membrane shall now sustain the weight w of the spherical shell, that transforms in the weight w^* on the ellipsoid setting

$$w^* r_1(\varphi)d\varphi r(\varphi)d\theta = wRd\beta r_s d\theta \;;\; r_s = R\sin\beta \tag{13}$$

whence

$$w^* = wR^2\frac{1}{br_2(\varphi)} \tag{14}$$

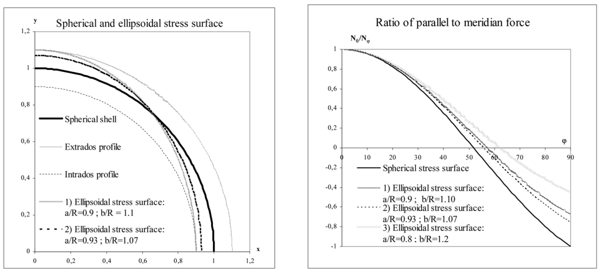
The equilibrium along the outward normal to the (ellipsoidal) membrane yields

$$\frac{N_\varphi(\varphi)}{r_1(\varphi)} + \frac{N_\theta(\varphi)}{r_2(\varphi)} = p_n(\varphi) = -w^*\cos\varphi = -w\frac{R^2}{br_2(\varphi)}\cos\varphi \tag{15}$$

and

$$N_\theta(\varphi) = -\left(w\frac{R^2}{b}\cos\varphi + N_\varphi(\varphi)\frac{r_2(\varphi)}{r_1(\varphi)}\right) \tag{16}$$

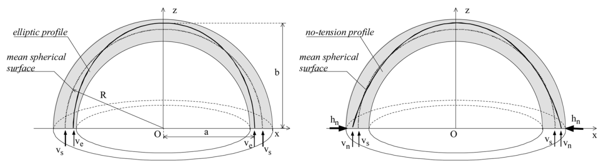
Ellipsoidal stress surface can be active in order to mitigate tension hoop stresses, possibly after some fractures have opened (Fig. 13b), that can be considered physiological if masonry has some degree of ductility in the parallel direction, as in the friction strength mechanism illustrated in Sec. 2. In Fig. 14a various membrane stress surfaces are plotted, with different ratios *a/b*. Note that such surfaces make sense provided that they remain included in the thickness of the spherical shell, i.e. if $t \geq 2(R-a)$ and $t \geq 2(b-R)$, with $b \geq a$. The plots in Fig. 14b prove that the ratio of the parallel to the meridian normal force can be mitigated, and also be near 0.4 and smaller, with increasing the ratio *b/a*. Consider that both in the spherical and in the elliptic membranes, the stress surface is a complete semi-ellipsoid, with φ = 90° at y = 0, so that the equilibrium solutions do not require any thrust force at the bottom support y = 0.



a) b)

Fig. 14. a) Ellipsoidal membrane surfaces for different ratios of the ellipse radii a and b to the radius R of the spherical dome; b) Ratio of Nq to Nj for different shapes of the elliptic profile

Anyway, it has been proved by [12] that a membrane surface included in the thickness of the dome can be found without hoop tension, provided that a adequate counter-thrust force can be exerted at the bottom of the dome. In Fig. 15a it is illustrated how the spherical and elliptic membranes only transfer vertical actions on the basement, v_s and v_e respectively, while a no-tension profile requires that the base support can support a horizontal force h_n (Fig. 15b).



a) b)

Fig. 15. No-thrust and no-tension stress surfaces: a) The basement of the dome is not subject to thrust action, but lower parallel lines are under tension; b) If a no-tension solution is adopted, the support of the dome is subject to a horizontal thrust force. Tension in the parallel lines is transferred to the basement

3.3. The effect of "masonry apparatus" on the statics of vaults. An help to intuition.

Reading masonry texture in a vault can help in understanding its equilibrium asset. The first element is indeed its geometry, a cross vault yields a equilibrium pattern different than a barrel vault, and so on. But a double-curvature surface, apart from its particular conception is anyway a highly hyperstatic system, and the equilibrium is never uniquely determinate. So the way the stones are jointed all together is a key to understand what equilibrium path would stresses run through, and/or what path would the builder have preferred to drive the vault to accommodate in.

So, consider for instance the two vaults in Fig. 16a and in Fig. 16b, having the same geometry, but in vault a) the mortar rows are parallel to the base perimeter, while in the vault b) the mortar rows are normal to the perimeter. The postulate is that compression normal to the mortar rows is the preferred equilibrium path for the vault, and that this is the tool for the original builder to steer the vault into a (his) objective static asset. If the preferred direction for compression is normal to the perimeter, it is expected that compression acts along the arrows drawn in Figs. 3.8, a) and b), so that the vault gains a tensile capacity in the direction orthogonal to the arrows. It is easy to understand that this produces an effect on the thrust the vault exerts on the base supports. Consider in fact that in both cases the vault is made by four gores. In the case a) compression is directly transferred to the sides of the basement, while lateral dilatation and the diffusion of stresses to the corners is contrasted by internal tensile strength; so two opposite gores tend to directly sustain each other, and the distribution of the horizontal thrust force tends to concentrate towards the middle of the sides (Fig. 16c). By contrast, in case b) compression is active in the direction parallel to the base sides, and the gores tend to support each other along the diagonal lines, while the orthogonal dilatation and diffusion of stress are now contrasted by tensile strength in the direction orthogonal to the sides; so all forces tend to converge in the corners, and the distribution of the horizontal thrust force tends to concentrate to the corners (Fig. 16d). In other words, by acting on the masonry apparatus it is possible that, with the same geometry, a structure may be realized that works like a cloister vault rather than like a groin vault or viceversa. Which means that it may be not wise to analyze the statics of a vault only on the basis of its geometry. Anyway, a skilled design of apparatus is also a tool to build vaults without formworks [13].

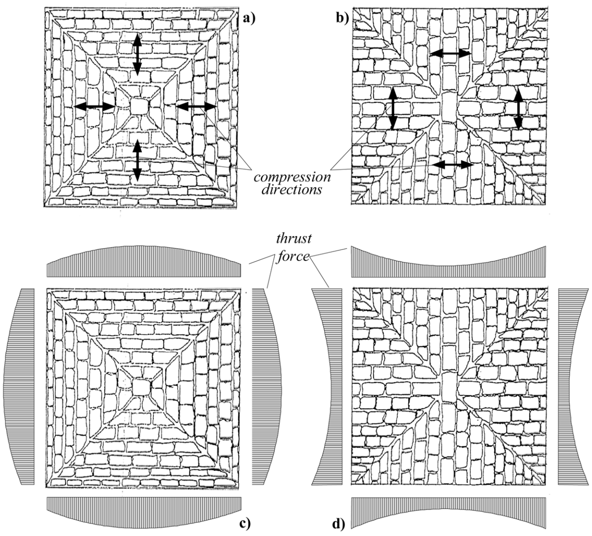


Fig. 16. Influence of the vault apparatus on the static behaviour of vaults. The difference in the apparatus in Figs. a) and b) yields a different equilibrium pattern and a different distribution of the thrust force as in Figs. c) and d)

4. MASONRY AS A NO-TENSION MATERIAL

In the previous section it has been recognized that in some cases a tensile capacity along some direction can be attributed to masonry. Anyway Eq. (1) is conditioned by the implicit assumption that σ'_{ox} is not larger than σ'_{ob}, the tensile strength of single bricks. If in any time in the life of the structure the stress s exceeds this limit, the bricks crack, and the strength decays to zero, with a brittle behaviour. On the other side, there is no doubt that the prevalent feature that characterizes masonry structures, and makes them dissimilar from modern concrete and steel structures, is quite definitely their intrinsic inability to resist tensile stresses. So, it is natural that the material model, that is intended to be an "analogue" of real masonry, cannot resist tensile stress, but, possibly, behaves elastically under pure compression.

No-Tension solutions for masonry structures are however a very significant reference point and a powerful tool for reliable structural assessment, for many reasons. The first reason is that the NT model is a stable behaviour, poorly subject to uncertainty and aging. Tensile strength is in any case small, uncertain, highly variable in the mass of a structure, not durable in time and so on; anyway neglecting tensile strength leads to a safe assessment. In other words, no doubt that the NT model is a simplified behaviour, that in some cases does not give account of some surprisingly good performance of masonry buildings, but it is also true that if a masonry structure does not pass through a NT check it remains a suspect structure.

In the following the basics for the foundation of a NT material theory are illustrated, and the relevant principles for structural analysis, mainly identified in the classic energy theorems, suitably adapted to the material at hand, are formulated.

In (apparently) simple cases, closed-form solutions can be obtained, or, at least, the solution process can be prepared after a preliminary screening of the equilibrium scenario.

4.1. The standard No-Tension material

In a NT solid the equilibrium against external loads is required to be satisfied by *admissible* stress fields, which imply pure compression everywhere in the solid. Compatibility of the strain field can be ensured by superposing to the elastic strain field an additional *fracture* field, that does not admit contraction in any point and along any direction; that is to say that the stress tensor σ must be negative semi-definite everywhere in the solid, while the fracture strain field ε_f is required to be positive semi-definite.

The material shall, hence, satisfy the following conditions

$$\begin{gathered}\boldsymbol{\varepsilon} = \boldsymbol{\varepsilon}_e + \boldsymbol{\varepsilon}_f = \mathbf{C}\boldsymbol{\sigma} + \boldsymbol{\varepsilon}_f \\ \text{Semi-Definite}\begin{cases}\boldsymbol{\varepsilon}_f \ \text{Positive} \\ \boldsymbol{\sigma} \ \text{Negative}\end{cases} \rightarrow \begin{cases}\varepsilon_{fa} \geq 0 \\ \sigma_a \leq 0\end{cases} \quad \forall a \in r_{\mathrm{a}} \\ \boldsymbol{\sigma} \cdot \boldsymbol{\varepsilon}_f \leq 0\end{gathered} \tag{17}$$

where r_a is the set of directions through the generic point in the solid, a is one of such directions, ε_f is the fracture strain that is assumed to superpose to the elastic strain ε_e in order to anneal tensile stresses if possible, and C denotes the tensor of elastic constants. Consider moreover that on every elementary surface with normal a, if ε_{fa} is strictly positive σ_a must be zero; by contrast if σ_a is strictly negative, ε_{fa} must be zero. If σ_o is the stress tensor in the point actually associated with fractures ε_f, it follows that

$$\boldsymbol{\sigma}_o \cdot \boldsymbol{\varepsilon}_f = 0 \tag{18}$$

The material admissibility conditions for strain and stress reported in Eq.(1) can be synthetically referred to by the set of inequalities $\mathbf{h}_\varepsilon(\varepsilon_f) \geq \mathbf{0}$ and $\mathbf{h}_\sigma(\sigma) \geq \mathbf{0}$ respectively.

As a consequence of Eqs. (17) and (18), the classical Drucker's rule holds for the fracture strain. With reference to the admissible domains quoted in Eq. (4.1), the normality Drucker's law for no-tension material can thus be written as

$$(\boldsymbol{\sigma}-\boldsymbol{\sigma}_\mathbf{o})\cdot\boldsymbol{\varepsilon}_f \leq 0 \qquad \forall\boldsymbol{\sigma}\in\Sigma \tag{19}$$

where Σ is the set of admissible stress tensors and σ is any admissible stress state other than the effective one σ_o.

4.2. Limit Analysis and fundamental theorems

Let consider the body and surface forces, **F** acting on volume V and **p** acting on the free surface S_p, the displacement field **u**, the imposed displacement field $\mathbf{u}_o$ characterizing the constrained part of the solid surface S_u, the above mentioned strain field $\varepsilon = \varepsilon_e + \varepsilon_f = \mathbf{C}\,\sigma + \varepsilon_f$, the stress field σ.

As clear from the above, fracture strains ε_f can be developed at the considered point only if the stress situation can be represented by a stress tensor σ laying on the surface of the material admissibility domain, which is defined for NT bodies by $\mathbf{h}_\sigma(\sigma) \leq \mathbf{0}$; obviously if some fracture does exist, it is developed according to the NT material inequalities $\mathbf{h}_\varepsilon(\varepsilon_f) \geq \mathbf{0}$.

4.2.1. General setup

Denoting by U the set of possible displacement fields, the class of *fracture admissible mechanisms* is defined by the subset U_f of U containing displacement fields $\mathbf{u}_f$ that are directly compatible with fracture strains ε_f apart from any elastic strain field

$$\boldsymbol{\varepsilon}_f = \nabla\,\mathbf{u}_f \;\;;\;\; \mathbf{h}_\varepsilon(\boldsymbol{\varepsilon}_f) \geq \mathbf{0} \tag{20}$$

$$U_f = \{\mathbf{u}_f \in U : \mathbf{h}_\varepsilon(\nabla\mathbf{u}_f) \geq \mathbf{0}\} \tag{21}$$

Collapse mechanisms can be defined as fracture admissible mechanisms $\mathbf{u}_f$ such that the mechanical work developed by the applied loads (**p**,**F**) is positive; this condition is analytically expressed by the inequality

$$\int_{S_p}\mathbf{p}\cdot\mathbf{u}_f\,dS + \int_V \mathbf{F}\cdot\mathbf{u}_f\,dV > 0 \tag{22}$$

By the Principle of Virtual Work, a necessary condition for the existence of any admissible stress field σ equilibrating the applied loads is that

$$\int_{S_p}\mathbf{p}\cdot\mathbf{u}_f\,dS + \int_V \mathbf{F}\cdot\mathbf{u}_f\,dV = \int_V \boldsymbol{\sigma}\cdot\boldsymbol{\varepsilon}_f\,dV \leq 0 \;\; \forall\mathbf{u}_f \in U_f \tag{23}$$

After Eq. (23) one can enounce the "Kinematical Theorem" of Limit Analysis for NT bodies: *if any collapse mechanism exists under the applied loads, no solution can exist for the equilibrium of the NT solid*. In other words: *If any collapse mechanism exists, the solid collapses.*

On the other side, *statically admissible stress fields* σ can be defined as tensor fields equilibrating the applied loads and satisfying admissibility conditions, i.e. $\mathbf{h}_\sigma(\sigma) \leq \mathbf{0}$ or $\sigma\in\Sigma$, where Σ is the admissible domain, everywhere in the solid. Assuming that under the load pattern (**p**,**F**) a statically admissible stress field σ exists for any mechanism $\mathbf{u}_f$, after Eq. (23) one gets

$$\int_{S_p}\mathbf{p}\cdot\mathbf{u}_f\,dS + \int_V \mathbf{F}\cdot\mathbf{u}_f\,dV \leq 0 \;\; \forall\mathbf{u}_f \in U_f \tag{24}$$

One can, thus, enounce the "Static Theorem" of Limit Analysis for NT bodies: *if under the applied loads any statically admissible stress field σ exists, no collapse mechanism exists and the structure cannot collapse*.

4.2.2. The one-multiplier load pattern. The safety factor

Let assume the applied loads as given by the sum of a fixed component ($\mathbf{F}_o$, $\mathbf{p}_o$) and a variable component ($s\mathbf{F}_v$, $s\mathbf{p}_v$) depending on the value assumed by the multiplier s (actually one thus assumes that only the portion $\mathbf{F}_v$, $\mathbf{p}_v$, may be destabilizing and should be controlled)

$$\begin{cases}\mathbf{F} = \mathbf{F}_o + s\mathbf{F}_v = \mathbf{0} & \text{in } V\\ \mathbf{p} = \mathbf{p}_o + s\mathbf{p}_v & \text{on } S_p\end{cases} \tag{25}$$

and let define two fundamental classes of load multipliers s for NT bodies: the class of *statically admissible multipliers* β and the class of *kinematically sufficient multipliers* γ. After denoting by α_n the unit outgoing vector normal to the surface S_p, load multipliers β are defined to be statically admissible if the following relations hold

$$\begin{cases}\operatorname{div}\boldsymbol{\sigma}^\beta + \mathbf{F}_o + \beta\mathbf{F}_v = \mathbf{0} & \text{in } V\\ \boldsymbol{\sigma}^\beta\,\boldsymbol{\alpha}_n = \mathbf{p}_o + \beta\mathbf{p}_v & \text{on } S_p\end{cases} \tag{26}$$

$$\mathbf{h}_\sigma(\boldsymbol{\sigma}^\beta) \leq \mathbf{0} \tag{27}$$

that is to say, if a stress field σ^β exists equilibrating the applied loads with $s = \beta$ and satisfying the NT material admissibility conditions. A stress field satisfying Eqs (26) and (27) is qualified as *statically admissible*.

On the other side, load multipliers γ are defined to be *kinematically sufficient* if the following relations hold

$$\begin{cases}\boldsymbol{\varepsilon}_f^\gamma = \nabla\mathbf{u}_f^\gamma & \text{in } V\\ \mathbf{u}_f^\gamma = \mathbf{0} & \text{on } S_u\end{cases} \tag{28}$$

$$\mathbf{h}_\varepsilon(\boldsymbol{\varepsilon}_f) \geq \mathbf{0} \tag{29}$$

$$\int_V \mathbf{F}_o\cdot\mathbf{u}_f^\gamma\,dV + \int_{S_p}\mathbf{p}_o\cdot\mathbf{u}_f^\gamma\,dS + \gamma\int_V \mathbf{F}_v\cdot\mathbf{u}_f^\gamma\,dV + \gamma\int_{S_p}\mathbf{p}_v\cdot\mathbf{u}_f^\gamma\,dS > 0 \tag{30}$$

(with ∇ the symmetrical gradient operator), that is to say, if any displacement field $\mathbf{u}_f^\gamma$ exists (a *collapse mechanism*) directly compatible with a NT admissible fracture strain ε_f^γ apart from any elastic strain field, and such that the condition stated by Eq.(30) is also satisfied. It is understood that the body is stable under the basic load pattern ($\mathbf{F}_o$, $\mathbf{p}_o$), and that Eq. (30) cannot be satisfied by any fracture strain field for $\gamma = 0$. In other terms it is assumed that the basic loads are suitably chosen in way that they cannot produce collapse.

Extensions to NT continua of the fundamental *static and kinematic theorems* of Limit Analysis allow individuating the value $\bar{s}$ of the load multipliers s, limiting the loading capacity of the body.

On the basis of the *static theorem*, one can state that "*the collapse multiplier* $\bar{s}$ *represents the maximum of the statically admissible multipliers* β"

$$\bar{s} = \max\{\beta \in B_o\} \tag{31}$$

where B_o is the class of statically admissible multipliers.

On the basis of the *kinematic theorem*, one can state that "*the collapse multiplier* $\bar{s}$ *represents the minimum of the kinematically sufficient multipliers* γ"

$$\bar{s} = \min\{\gamma \in \Gamma_o\} \tag{32}$$

where Γ_o is the class of kinematically sufficient multipliers.

Thereafter, by means of the static theorem, one can search for the collapse multiplier by implementing the problem

$$\text{Find}: \max_{\beta,\sigma^\beta}\{\beta\}$$

$$\text{Sub}\begin{cases}\operatorname{div}\boldsymbol{\sigma}^\beta + \mathbf{F}_o + \beta\mathbf{F}_v = \mathbf{0} & \text{in } V\\ \boldsymbol{\sigma}^\beta\boldsymbol{\alpha}_n = \mathbf{p}_o + \beta\mathbf{p}_v & \text{on } S_p\end{cases} \text{ and } \mathbf{h}_\sigma\left(\boldsymbol{\sigma}^\beta\right)\le\mathbf{0} \tag{33}$$

Or otherwise, by means of the kinematic theorem, by solving the problem

$$\text{Find}: \min_{\gamma,\varepsilon_f^\beta,\mathbf{u}_f^\beta}\{\gamma\} \quad \text{Sub}\begin{cases}\boldsymbol{\varepsilon}_f^\gamma = \nabla\mathbf{u}_f^\gamma & \text{in } V\\ \mathbf{u}_f^\gamma = \mathbf{0} & \text{on } S_u\end{cases} \text{ and}$$

$$\begin{cases}\mathbf{h}_\varepsilon\left(\boldsymbol{\varepsilon}_f^\gamma\right)\ge\mathbf{0}\\ \int_V \mathbf{F}_o\cdot\mathbf{u}_f^\gamma dV + \int_{S_p}\mathbf{p}_o\cdot\mathbf{u}_f^\gamma dS + \gamma\int_V\mathbf{F}_v\cdot\mathbf{u}_f^\gamma dV + \gamma\int_{S_p}\mathbf{p}_v\cdot\mathbf{u}_f^\gamma dS > 0\end{cases} \tag{34}$$

4.3. Variational principles for the NT equilibrium problem

Analysis of no-tension structures proves that stress, strain and displacement fields obey extremum principles of the basic energy functionals. Therefore the solution displacement and fracture strain fields are found as the constrained minimum of the *Potential Energy* functional, under the condition that the fracture field is positively semi-definite at any point. In other words, if ε and **u** are respectively the strain and the displacement fields such that

$$\boldsymbol{\varepsilon} = \nabla\mathbf{u} \tag{35}$$

and **p**, **F** are the surface tractions and the body forces, it is possible to write down the Total Potential Energy (TPE) functional

$$\begin{aligned}\mathbf{E}(\mathbf{u},\boldsymbol{\varepsilon}_f) &= \frac{1}{2}\int_V(\nabla\mathbf{u}-\boldsymbol{\varepsilon}_f)[\mathbf{D}(\nabla\mathbf{u}-\boldsymbol{\varepsilon}_f)]dV -\\ &- \int_{S_p}\mathbf{p}\cdot\mathbf{u}dS - \int_V\mathbf{F}\cdot\mathbf{u}dV = \mathrm{L}(\mathbf{u},\boldsymbol{\varepsilon}_f)+\mathrm{P}(\mathbf{u})\end{aligned} \tag{36}$$

with **D** the inverse tensor of **C**. The TPE functional $\mathbf{E}(\mathbf{u}, \varepsilon_f)$ is made up by two terms, expressing the energy stored in the body $L(\mathbf{u}, \varepsilon_f)$ and the opposite of the work made by the applied loads $P(\mathbf{u})$. It can be proved that the solution $\mathbf{u}_o$, ε_{fo} satisfies the following condition

$$\mathbf{E}(\mathbf{u}_o,\boldsymbol{\varepsilon}_{fo}) = \min_{\substack{\mathbf{u}\in U\\ \boldsymbol{\varepsilon}_f\in\Phi}}\mathbf{E}(\mathbf{u},\boldsymbol{\varepsilon}_f) = \mathbf{E}_o \tag{37}$$

which is the minimum of the Potential Energy, conditioned upon admissibility of the fracture strain, with Φ the set of admissible fracture fields. Despite the quadratic functional $L(\mathbf{u}, \varepsilon_f)$ is positive definite, the minimum may be not unique if some mechanism exists such that $P(\mathbf{u}) = 0$.

The stress field can be found, in turn, as the constrained minimum of the Complementary Energy (CE) functional, under the condition that the stress field is in equilibrium with the applied loads and compressive everywhere. In other words, let

$$\mathbf{S}(\boldsymbol{\sigma}) = \frac{1}{2}\int_V\boldsymbol{\sigma}\cdot\mathbf{C}\boldsymbol{\sigma}dV - \int_{S_u}\mathbf{T}\cdot\mathbf{u}_o dS = L_c(\boldsymbol{\sigma})+R(\boldsymbol{\sigma});\quad \mathbf{T}=\boldsymbol{\sigma}\boldsymbol{\alpha}_n \tag{38}$$

be the CE functional S (σ) defined on the set Σ_o of the admissible stress fields ($\Sigma_o \subseteq \Sigma$) in equilibrium with the applied loads, with $L_c(\sigma)$ the complementary energy stored in the body and $R(\sigma)$ the work by the reactions times the settlements of the constrained points. It can be proved that, if σ_o is the solution stress field, the following condition holds

$$\mathbf{S}(\boldsymbol{\sigma}_o) = \min_{\sigma\in\Sigma_o}\mathbf{S}(\boldsymbol{\sigma}) = \mathbf{S}_o \tag{39}$$

Eq.(39) expresses the compatibility condition on the solution stress field, i.e. the constrained minimum of S (σ) yields the stress field σ_o such that the elastic strains $\mathbf{C}\sigma_o$ can be made compatible with a continuous displacement field, by the superposition of a fracture strain field. Since $\sigma \cdot \mathbf{C}\sigma$ is positive definite in Σ, the solution is unique.

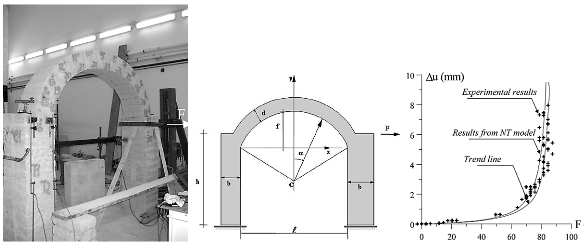
4.4. Convexity of the energy functionals and Limit Analysis as a tool for existence of the solution

It is easy to prove that the Total Potential Energy and the Complementary Energy functionals are both defined on convex sets. The respective sets of definition are: i) the space of couples $(\mathbf{u},\varepsilon_f)$, with **u** a displacement vector function compatible with the external constraints and ε_f a semi-positively valued tensor field; ii) the space of semi-negatively valued tensor fields in equilibrium with the applied loads. Because of convexity, both minima exist if the respective definition sets, $U \times \Phi$ on one side and Σ_o on the other side, are not empty. For Σ_o to be not empty it is necessary and sufficient that the structure is under the collapse threshold; in this case a unique minimal point exists for S (σ). U and Φ are intrinsically not empty, in that the first is the set of three-components vector fields and the second is the space of semi-positive definite 3rd-order tensor fields. The displacement/fracture solution may be not unique if a mechanism exists such that the external work is zero. Anyway, if any collapse mechanism exists, E $(\mathbf{u},\varepsilon_f)$ diverges, and the minimum does not exist.

It can be concluded that the solution of the NT equilibrium problem exists iff the structure is under the collapse threshold.

4.5. The NT solution for masonry arches and barrel vaults

NT solutions have been investigated since many years and many results have been produced, also yielding successful comparison with experimental results. As an example, consider the portal arch in Fig. 17a, that has been tested under an horizontal force acting on top of the right pillar, as in Fig. 17b. In Fig. 17c the comparison between the experimental and numerical results is plotted, proving a very good agreement of the NT theory with practice.



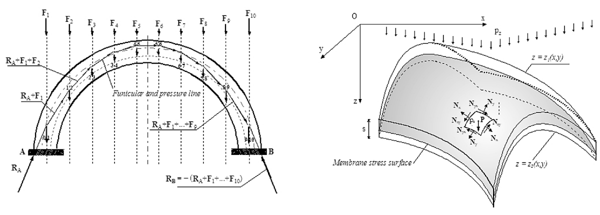
a) b) c)

Fig. 17. Experimental and mechanical model: a) The laboratory arch-portal tested under the horizontal force F; b) The mechanical NT model for calculations; c) Plot of numerical and experimental results

More details on NT material and structures can be found in [14, 15].

5. NO-TENSION MODEL FOR MASONRY – LIKE SHELLS AND DOMES

Equilibrium fields for No-Tension vaults can be built upon the assumption that a membrane stress-surface is considered, included in the profile of the vault, displaying compressive forces along all directions. The idea is not new, it was put forward by Heyman [11], but there are few doubts that it represents a powerful approach to search stresses in masonry vaults, as the 3D direct counterpart of the traditional historical method based on the funicular line of the loads in 2D structural problems (Fig. 18).



a) b)

Fig. 18. a) Arch equilibrium analysis: the pressure line; b) Vault equilibrium analysis: the membrane stress surface

As illustrated in Sec. 4, modelling a large variety of equilibrium stress fields is the preliminary step to find a final solution, yielding a credible pattern for the fracture distribution, in agreement with technical expectation, as it can be recognized for the two-span arch in Fig. 19

In the following an approach is outlined to identify membrane stress surfaces both responding to the requirements of stress admissibility and equilibrium with active loads.

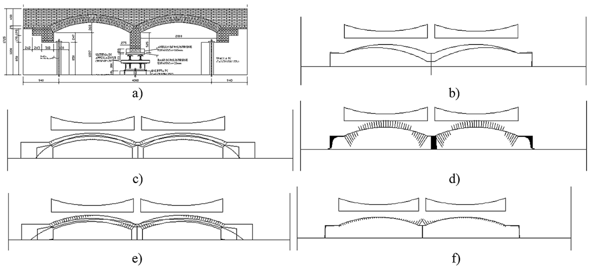


Fig. 19. Sample results from NT model. a) The arcade and the experimental set up; b) The arcade in the original configuration and with downward settlement of the central pier; c) Pressure line and fractures without settlement; d) Stresses without settlement; e) Pressure line and fractures with settlement; f) Stresses with settlement

5.1. Membrane 3D-equilibrium of the generic vault element

Let consider a membrane shell $z = z\ (x,y)$ subject to generic load components p_x, p_y, p_z , as shown in Fig. 20. In order to express local equilibrium conditions one should consider any single surface element under the action of loads and of membrane stresses, and write equilibrium equations along coordinate axes, as in Fig. 21.

After some algebra, the equilibrium equations in the x and y directions can be written in the form

$$\frac{\partial}{\partial x}\left(N_x \frac{\cos\varphi}{\cos\theta}\right) + \frac{\partial N_{xy}}{\partial y} = -p_x \frac{1}{\cos\varphi}\frac{1}{\cos\theta} \tag{40}$$

$$\frac{\partial N_{xy}}{\partial x} + \frac{\partial}{\partial y}\left(N_y \frac{\cos\theta}{\cos\varphi}\right) = -p_y \frac{1}{\cos\varphi}\frac{1}{\cos\theta} \tag{41}$$

Additionally, equilibrium along the z-direction yields

$$\begin{aligned}&\left(\frac{\partial N_x}{\partial x}\frac{1}{\cos\theta} + \frac{\partial N_{xy}}{\partial y}\frac{1}{\cos\varphi}\right)\sin\phi + \\ &+\left(\frac{\partial N_{xy}}{\partial x}\frac{1}{\cos\theta} + \frac{\partial N_y}{\partial y}\frac{1}{\cos\varphi}\right)\sin\theta = -p_z \frac{1}{\cos\varphi}\frac{1}{\cos\theta}\end{aligned} \tag{42}$$

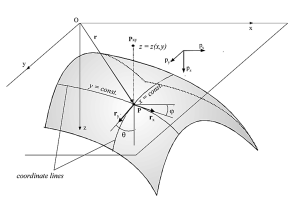


Fig. 20. The membrane surface

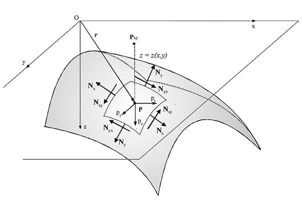


Fig. 21. Generic surface element of the membrane surface

5.2. Equivalence of the 3D-problem with the projected 2D-problem

Considering the projections $\overline{N}_x, \overline{N}_y, \overline{N}_{xy} = \overline{N}_{yx}$ of the membrane stresses onto the xy plane, as shown in Fig. 22

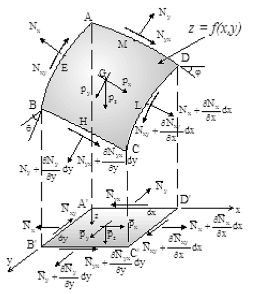


Fig. 22. Stresses and loads acting on a generic 3D-element of the vault mid-surface and their projection on the plane

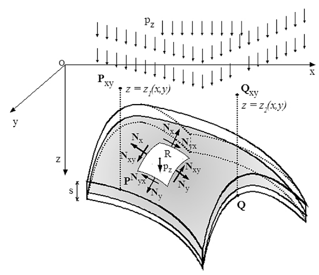


Fig. 23. The vault and its mid-surface under purely vertical loads

the following equations hold

$$\overline{N}_x = N_x \frac{\cos\varphi}{\cos\theta}, \quad \overline{N}_y = N_y \frac{\cos\theta}{\cos\varphi}, \quad \overline{N}_{xy} = N_{xy} \tag{43}$$

with

$$\tan\varphi = \frac{\partial z}{\partial x}, \qquad \tan\theta = \frac{\partial z}{\partial y}. \tag{44}$$

Moreover putting

$$p_x = \overline{p}_x \cos\theta\cos\varphi, \quad p_y = \overline{p}_y \cos\theta\cos\varphi \tag{45}$$

the equilibrium equations in function of the new variables can be written

$$\frac{\partial \overline{N}_x}{\partial x} + \frac{\partial \overline{N}_{xy}}{\partial y} + \overline{p}_x = 0; \qquad \frac{\partial \overline{N}_{xy}}{\partial x} + \frac{\partial \overline{N}_y}{\partial y} + \overline{p}_y = 0 \tag{46}$$

The z-equation, after some algebra, reduces to

$$\frac{\partial}{\partial x}\left(\overline{N}_x \frac{\partial z}{\partial x} + \overline{N}_{xy} \frac{\partial z}{\partial y} \right) + \frac{\partial}{\partial y}\left(\overline{N}_{xy} \frac{\partial z}{\partial x} + \overline{N}_y \frac{\partial z}{\partial y} \right) + \overline{p}_z = 0 \tag{47}$$

whence one gets

$$\overline{N}_x \frac{\partial^2 z}{\partial x^2} + 2\overline{N}_{xy} \frac{\partial^2 z}{\partial x \partial y} + \overline{N}_y \frac{\partial^2 z}{\partial y^2} = -\overline{p}_z + \overline{p}_x \frac{\partial z}{\partial x} + \overline{p}_y \frac{\partial z}{\partial y} \tag{48}$$

So the membrane equilibrium, apart from Eq. (48), becomes analogous to the problem of a plane panel.

5.3. Reduction of equilibrium conditions by means of the stress function

In most cases, it is advantageous to introduce a *stress function* $\Psi(x, y)$ that reduces the 3 equilibrium equations to one second-order equation as follows. By analogy with the panel problem, the equilibrium conditions in the x and y directions are identically satisfied if one puts

$$\overline{N}_x = \frac{\partial^2 \Psi}{\partial y^2} - \int \overline{p}_x dx, \quad \overline{N}_y = \frac{\partial^2 \Psi}{\partial x^2} - \int \overline{p}_y dy, \quad \overline{N}_{xy} = \overline{N}_{yx} = -\frac{\partial^2 \Psi}{\partial x\, \partial y}. \tag{49}$$

Then the third equilibrium equation (48) turns into

$$\frac{\partial^2 \Psi}{\partial x^2}\frac{\partial^2 z}{\partial y^2} - 2\frac{\partial^2 \Psi}{\partial x\, \partial y}\frac{\partial^2 z}{\partial x \partial y} + \frac{\partial^2 \Psi}{\partial y^2}\frac{\partial^2 z}{\partial x^2} = = -\overline{p}_z + \overline{p}_x \frac{\partial z}{\partial x} + \overline{p}_y \frac{\partial z}{\partial y} + \frac{\partial^2 z}{\partial x^2}\int \overline{p}_x dx + \frac{\partial^2 z}{\partial y^2}\int \overline{p}_y dy \tag{50}$$

The solution of the problem is, thus, reduced to the determination of the stress function. If $p_x = p_y = 0$ (e.g. the vault sustains only gravitational loads), the latter equation simplifies to:

$$\frac{\partial^2 \Psi}{\partial x^2}\frac{\partial^2 z}{\partial y^2} - 2\frac{\partial^2 \Psi}{\partial x\, \partial y}\frac{\partial^2 z}{\partial x \partial y} + \frac{\partial^2 \Psi}{\partial y^2}\frac{\partial^2 z}{\partial x^2} = -\overline{p}_z \tag{51}$$

5.4. Stress NT admissibility

Let consider a masonry vault with thickness s, subject only to vertical loads, as shown in Fig. 23 Since it is assumed that the masonry cannot resist tensile stresses, the internal forces have to satisfy the following admissibility conditions

$$N_x \le 0, \quad N_y \le 0, \quad N_{xy}^2 - N_x N_y \le 0 \tag{52}$$

A solution can be attempted searching for a membrane surface $z = z(x,y)$ completely internal to the mass of the vault, and such to resist the downward (i.e. positive) load p_z by purely compressive internal forces. The first condition is expressed by the inequalities (the *inclusion* condition)

$$z_1(x, y) \le z(x, y) \le z_2(x, y) \tag{53}$$

where $z_1(x,y)$ and $z_2(x,y)$ are the surfaces identifying the upper and lower profiles of the vault, respectively.

Definitively, setting the problem in plane variables and assuming that only vertical loads act, the equilibrium is expressed by Eqs. (49) with $p_x = p_y = 0$ and by Eq. (51), whilst admissibility conditions are given by

$$\left.\begin{matrix} N_x \le 0 \\ N_y \le 0 \\ N_{xy}^2 - N_x N_y \le 0 \end{matrix}\right\rangle \Leftrightarrow \left\langle\begin{matrix} \overline{N}_x \le 0 \\ \overline{N}_y \le 0 \\ \overline{N}_{xy}^2 - \overline{N}_x \overline{N}_y \le 0 \end{matrix}\right. \tag{54}$$

Eqs. (54) are conditioned by Eq. (53), according to which the stress membrane shell of the vault should be kept within the vault's profiles.

5.5. Simple coupling of stress equilibrium with admissibility

So, the key of the problem is how the stress function $\Psi(x,y)$ combines with the membrane function $z(x,y)$. It is interesting to note that if one takes

$$\Psi(x,y) = -k\, z(x,y), \; k > 0 \tag{55}$$

by substitution in Eq.(49), one gets

$$\overline{N}_x = -k \frac{\partial^2 z}{\partial y^2}, \quad \overline{N}_y = -k \frac{\partial^2 z}{\partial x^2}, \quad \overline{N}_{xy} = \overline{N}_{yx} = k \frac{\partial^2 z}{\partial x\, \partial y}. \tag{56}$$

Coupling with admissibility conditions, one gets

$$\left.\begin{array}{r}\overline{N}_x \le 0\\ \overline{N}_y \le 0\\ \overline{N}_{xy}^2-\overline{N}_x\overline{N}_y\le 0\end{array}\right\}\Leftrightarrow\left\{\begin{array}{l}\left.\begin{array}{l}\dfrac{\partial^2 z}{\partial y^2}\ge 0\\ \dfrac{\partial^2 z}{\partial x^2}\ge 0\end{array}\right\}\\ \dfrac{\partial^2 z}{\partial x^2}\dfrac{\partial^2 z}{\partial y^2}-\left[\dfrac{\partial^2 z}{\partial x\partial y}\right]^2=\mathcal{H}_z(x,y)\ge 0\end{array}\right. \tag{57}$$

$$\forall(x,y)\in \mathrm{X}$$

with $\mathcal{H}_z(x,y)$ the Hessian determinant of the z function, and X the horizontal projection of the vault. On the other side, substitution of Eq. (55) in the z-equilibrium Eq. (51) yields

$$\mathcal{H}_z(x,y)=\frac{\partial^2 z}{\partial x^2}\frac{\partial^2 z}{\partial y^2}-\left[\frac{\partial^2 z}{\partial x\,\partial y}\right]^2=\frac{\overline{p}_z}{Q}\ge 0;\quad Q=2k \tag{58}$$

with Q the thrust basement factor. From Eqs (57) one can deduct that $z(x,y)$ is a concave function. One can also observe that in a masonry vault. equilibrium may coincide with admissibility. This result means that a single joint equation is able to account at the meanwhile for both equilibrium and admissibility, also giving a scientific demonstration of the easiness of building masonry vaults, due to the almost immediate intuition of equilibrium against applied load, which is able to assure also admissibility.

Eq. (58) represents the simplest form of the well known Monge-Ampère equation [16], and quite clearly plays a key role in the statics of NT vaults [17].

5.6. General coupling of stress equilibrium with admissibility

More in general consider the position

$$\Psi(x,y) = F[z(x,y)] \tag{59}$$

Eq.s (49) turn to

$$\overline{N}_x=\frac{\partial^2 F}{\partial z^2}\left[\frac{\partial z}{\partial y}\right]^2+\frac{\partial F}{\partial z}\frac{\partial^2 z}{\partial y^2};\ \overline{N}_y=\frac{\partial^2 F}{\partial z^2}\left[\frac{\partial z}{\partial x}\right]^2+\frac{\partial F}{\partial z}\frac{\partial^2 z}{\partial x^2}$$
$$\overline{N}_{xy}=\overline{N}_{yx}=-\left(\frac{\partial^2 F}{\partial z^2}\frac{\partial z}{\partial y}\frac{\partial z}{\partial x}+\frac{\partial F}{\partial z}\frac{\partial^2 z}{\partial x\,\partial y}\right). \tag{60}$$

and Eqs. (54) yield

$$\left.\begin{array}{r}\overline{N}_x \le 0\\ \overline{N}_y \le 0\\ \overline{N}_{xy}^2-\overline{N}_x\overline{N}_y\ge 0\end{array}\right\}\Leftrightarrow$$

$$\Leftrightarrow\left\{\begin{array}{l}\left.\begin{array}{l}\dfrac{\partial^2 F}{\partial z^2}\left[\dfrac{\partial z}{\partial y}\right]^2+\dfrac{\partial F}{\partial z}\dfrac{\partial^2 z}{\partial y^2}\le 0\\ \dfrac{\partial^2 F}{\partial z^2}\left[\dfrac{\partial z}{\partial x}\right]^2+\dfrac{\partial F}{\partial z}\dfrac{\partial^2 z}{\partial x^2}\le 0\end{array}\right\}\\ \left(\dfrac{\partial^2 F}{\partial z^2}\left[\dfrac{\partial z}{\partial y}\right]^2+\dfrac{\partial F}{\partial z}\dfrac{\partial^2 z}{\partial y^2}\right)\left(\dfrac{\partial^2 F}{\partial z^2}\left[\dfrac{\partial z}{\partial x}\right]^2+\dfrac{\partial F}{\partial z}\dfrac{\partial^2 z}{\partial x^2}\right)+\\ -\left[\dfrac{\partial^2 F}{\partial z^2}\dfrac{\partial z}{\partial y}\dfrac{\partial z}{\partial x}+\dfrac{\partial F}{\partial z}\dfrac{\partial^2 z}{\partial x\,\partial y}\right]^2\le 0\end{array}\right. \tag{61}$$

while the z-equilibrium Eq. (51) is

$$2\frac{\partial F}{\partial z}\left(\frac{\partial^2 z}{\partial x^2}\frac{\partial^2 z}{\partial y^2}-\frac{\partial^2 z}{\partial x\,\partial y}\frac{\partial^2 z}{\partial x\,\partial y}\right)+$$
$$+\frac{\partial^2 F}{\partial z^2}\left[\left(\frac{\partial z}{\partial x}\right)^2\frac{\partial^2 z}{\partial y^2}-2\frac{\partial z}{\partial y}\frac{\partial z}{\partial x}\frac{\partial^2 z}{\partial x\,\partial y}+\left(\frac{\partial z}{\partial y}\right)^2\frac{\partial^2 z}{\partial x^2}\right]=-\overline{p}_z \tag{62}$$

5.7. Problem solution in terms of stress and fractures by the complementary energy approach

Once membrane forces have been identified, stresses and consequently elastic energy can be easily calculated.

Therefore, after individuating the set of statically admissible membrane surfaces (in terms of admissibility and equilibrium, i.e. satisfying the set of inequalities in Eqs (40)-(41)-(42) and (52)), a possible approach in order to find the solution in terms of stresses is to set up a Complementary Energy problem to be formulated as a kind of extension to masonry vaults of the classical analogous energetic approach for linearly elastic structures, aiming at finding the fracture field that yields a compatible strain field when combined with elastic strains.

In order to undertake this approach the expression of the Complementary Energy embedded in a masonry vault element shall be evaluated. In case of NT assumption, one should consider that the generic element appears to be partially resistant; with reference to Fig. 24 one has the following stress and strain components in he membrane's plane

$$\left\{\begin{array}{l}\sigma_x(C_N)=\sigma_{mx}\left(1+\dfrac{e_r}{d_{G_r}}\right)=\dfrac{N_x}{3u}\left(1+\dfrac{e_r}{d_{G_r}}\right)\\ \sigma_y(C_N)=\sigma_{my}\left(1+\dfrac{e_r}{d_{G_r}}\right)=\dfrac{N_y}{3u}\left(1+\dfrac{e_r}{d_{G_r}}\right)\end{array}\right.$$
$$\left\{\begin{array}{l}\varepsilon_x(C_N)=\dfrac{1}{E}\left[\sigma_x(C_N)-\nu\sigma_y(C_N)\right]\\ \varepsilon_y(C_N)=\dfrac{1}{E}\left[\sigma_y(C_N)-\nu\sigma_x(C_N)\right]\end{array}\right. \tag{63}$$

where C_N is the point where the membrane surface intersects the normal to the middle surface of the vault, u is the distance from C_N to the compressed profile of the vault element, the neutral surface where stresses are null is at distance $3u$ from the compressed profile, the part of the vault element below (or above) the neutral surface is inert and possibly fractured, $A_{rx} = 3u ds_y$ and $A_{ry} = 3u ds_x$ are the reactive areas of the respective cross-sections, G_r is the centroid of the reactive part of the vault element, e_r is the distance of the membrane from G_r, and N_x, N_y are the normal forces per unit length on the element edges; $\sigma_{mx} = \sigma_x(G_r)$, $\sigma_{my} = \sigma_y(G_r)$ are the average compression stresses on respectively A_{rx} and A_{ry}

$$\sigma_{mx}=\sigma_x(G_r)=\frac{N_x}{3u}\quad;\quad\sigma_{my}=\sigma_y(G_r)=\frac{N_y}{3u} \tag{64}$$

Eqs. (63) and (64) after some algebra yield

$$\begin{cases} \sigma_x(C_N) = \dfrac{N_x}{3u}\left(1+\dfrac{e_r}{d_{G_r}}\right) \\ \sigma_y(C_N) = \dfrac{N_y}{3u}\left(1+\dfrac{e_r}{d_{G_r}}\right) \end{cases}$$

$$\begin{cases} \varepsilon_x(C_N) = \dfrac{1}{3uE}\left(1+\dfrac{e_r}{d_{G_r}}\right)\left(N_x - \nu N_y\right) \\ \varepsilon_y(C_N) = \dfrac{1}{3uE}\left(1+\dfrac{e_r}{d_{G_r}}\right)\left(N_y - \nu N_x\right) \end{cases} \tag{65}$$

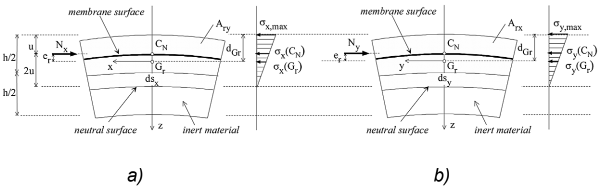


Fig. 24. Cross sections of the generic element of the vault and normal stresses distribution: a) Cross-section xz, b) Cross-section yz

When neglecting the curvatures of the element and the shear stress component, the elementary elastic work is

$$dL_e = \frac{1}{2}\left[N_x\varepsilon_x(C_N) + N_y\varepsilon_y(C_N)\right]ds_x ds_y =$$
$$= \frac{1}{6uE}\left(1+\frac{e_r}{d_{G_r}}\right)\left(N_x^2 - 2\nu N_x N_y + N_y^2\right)ds_x ds_y \tag{66}$$

and so is the elastic energy stored in the element dL = dL_e. The elastic energy over the whole vault is

$$L_e = L = \int_S dL =$$
$$= \frac{1}{6E}\int_S \frac{1}{6uE}\left(1+\frac{e_r}{d_{G_r}}\right)\left(N_x^2 - 2\nu N_x N_y + N_y^2\right)ds_x ds_y \tag{67}$$

where S is the mean surface of the vault. In Eq. (67) all the entities in the integrand function are dependent by the point coordinates through the membrane function $z(x,y)$.

The final Complementary Energy functional S expression is given by adding to the elastic energy term L, the energy related to the work developed by the constraint reactions R, as S $=L+R$.

The solution in terms of stresses shall be then searched for by numerically implementing the minimization of the Complementary Energy functional over admissible z-functions, i.e. under the condition that the solution itself is respectful of the above individuated equilibrium and admissibility equations.

6. SOLUTION OF NT EQUILIBRIUM. THE VAULTS OF TRANSLATION

After membrane forces have been identified, stresses and consequently elastic energy are easily calculated.

As shown in the previous paragraph, a possible approach to the problem consists in hypothesizing a shape of the stress function $\Psi(x,y)$ a priori satisfying equilibrium and material admissibility, and, thereafter, deducing the related membrane function $z(x,y)$, and, therefore, stress distribution; in some cases this approach allows to identify solutions fitting some vault typologies; in the following, the case is presented of vaults of translation, such as the simple case of the barrel vault with indefinite length and the -less simple- barrel vault with constraints at its longitudinal extremities, which requires a rather more complex treatment.

6.1. The indefinite length barrel vault

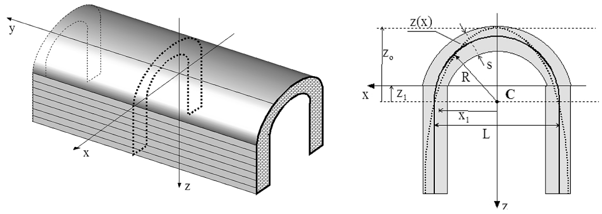


Fig. 25. The indefinite barrel vault and its cross-section

With reference to Fig. 25, consider the stress function in the form

$$\Psi(x,y) = -kz(x,y) + f(y) + g(x) \tag{68}$$

whence

$$\frac{\partial^2\Psi(x,y)}{\partial x^2} = -k\frac{\partial^2 z(x,y)}{\partial x^2} + g''(x)\ ;\ \frac{\partial^2\Psi(x,y)}{\partial y^2} =$$
$$= -k\frac{\partial^2 z(x,y)}{\partial y^2} + f''(y)\ ;\ \frac{\partial^2\Psi(x,y)}{\partial x\,\partial y} = -k\frac{\partial^2 z(x,y)}{\partial x\,\partial y} \tag{69}$$

From Eq.(51), equilibrium turns into

$$-2k\frac{\partial^2 z(x,y)}{\partial x^2}\frac{\partial^2 z(x,y)}{\partial y^2} + 2k\left(\frac{\partial^2 z(x,y)}{\partial x\,\partial y}\right)^2 +$$
$$+ f''(y)\frac{\partial^2 z(x,y)}{\partial x^2} + g''(x)\frac{\partial^2 z(x,y)}{\partial y^2} = -\bar{p}_z(x,y) \tag{70}$$

If one considers

$$\bar{p}_z(x,y) = \bar{p}_z(x)\ ;\ z(x,y) = z(x) \tag{71}$$

Eq. (70) reduces to

$$f''(y)\frac{\partial^2 z(x)}{\partial x^2} = -\bar{p}_z(x) \tag{72}$$

which can be solved with

$$f''(y) = \text{const.} = -H;\ \ H \geq 0 \tag{73}$$

whence

$$z''(x)=\frac{\overline{p}_z(x)}{H}\geq 0$$

$$\overline{N}_x=\frac{\partial^2\Psi(x,y)}{\partial y^2}=f''(y)=-H\leq 0\ ,\ \overline{N}_y=\frac{\partial^2\Psi(x,y)}{\partial x^2}= \tag{74}$$

$$=-kz''(x)+g''(x)\ ,\ \overline{N}_{xy}=\frac{\partial^2\Psi(x,y)}{\partial x\,\partial y}=0$$

According to Eq. (56) the solution is admissible if $-kz''(x)+g''(x)\leq 0$. If one takes $-kz''(x)+g''(x)=0$ the previous Eqs (73) yield $N_y = 0$, and the equation

$$z''(x)=\frac{\overline{p}_z(x)}{H}\geq 0 \tag{75}$$

yields the funicular line of the active load, with H the relevant thrust, which is the well known solution for the barrel vault as a sequence of independent arches (Fig. 25).

6.2. The confined barrel vault

A second case can be obtained setting

$$\Psi(x,y)=ay^2\alpha(x)\quad ;\quad z(x,y)=\frac{\Psi_o(x,y)-\Psi(x,y)}{2k} \tag{76}$$

with the coefficient a such that $0 \leq aL \leq 1$, being L a length parameter, and $\alpha(x)$ obeying the equation

$$q\alpha(x)\alpha''(x)-\alpha'^2(x)=k\,\overline{p}_z(x)\geq 0\ ;\ q=1/2 \tag{77}$$

One should notice that $\alpha(x)$ is the function governing the internal forces in the membrane surface. In fact, from Eq.(49), one has

$$\overline{N}_x(x,y)=\frac{\partial^2\Psi(x,y)}{\partial y^2}=2a\alpha(x);\ \overline{N}_{xy}(x,y)=\frac{\partial^2\Psi(x,y)}{\partial x\partial y}=2ay\alpha'(x);$$
$$\overline{N}_y(x,y)=\frac{\partial^2\Psi(x,y)}{\partial x^2}=ay^2\alpha''(x) \tag{78}$$

$\Psi_o(x,y)$ is assigned in the form

$$\Psi_o(x,y)=G(x)y^2+Q(x) \tag{79}$$

with $G(x)$ and $Q(x)$ obeying the following differential equations

$$2a\alpha''(x)G(x)-8a\alpha'(x)G'(x)+2a\alpha(x)G''(x)y^2=8ka^2\overline{p}_z(x)$$
$$2a\alpha(x)Q''(x)=-2k\overline{p}_z(x) \tag{80}$$

The membrane surface remains identified by the equation:

$$z(x,y)=\frac{\Psi_o(x,y)-\Psi(x,y)}{2k}=\frac{G(x)y^2+Q(x)-ay^2\alpha(x)}{2k} \tag{81}$$

The initial values of $\alpha(x)$, $G(x)$, $Q(x)$ with the relevant derivatives, and the values of a and k remain undetermined and available to set additional constraints.

In fact, if the problem is symmetric in both directions <x> and <y>, the following conditions shall be added:

$$\left(\frac{\partial z}{\partial x}\right)_{x=-\ell/2}=z_x\left(-\frac{\ell}{2},y\right)=$$
$$=\frac{G'(-\ell/2)y^2+Q'(-\ell/2)-ay^2\alpha'(-\ell/2)}{2k}=z_1(y)\leq 0$$
$$\left(\frac{\partial z}{\partial x}\right)_{x=+\ell/2}=z_x\left(+\frac{\ell}{2},y\right)=$$
$$=\frac{G'(+\ell/2)y^2+Q'(+\ell/2)-ay^2\alpha'(+\ell/2)}{2k}=z_2(y)\geq 0 \tag{82}$$

$$z_2(y)=-z_1(y)\ \ \forall y\Rightarrow$$
$$\Rightarrow\begin{cases}G'(+\ell/2)+G'(-\ell/2)=a[\alpha'(+\ell/2)+\alpha'(-\ell/2)]\\ Q'(+\ell/2)=-Q'(-\ell/2)\end{cases} \tag{83}$$

still because of symmetry

$$\left(\frac{\partial z}{\partial x}\right)_{x=-\ell/2}=z_x\left(-\frac{\ell}{2},y\right)=$$
$$=\frac{G'(-\ell/2)y^2+Q'(-\ell/2)-ay^2\alpha'(-\ell/2)}{2k}=z_1(y)\leq 0$$
$$\left(\frac{\partial z}{\partial x}\right)_{x=+\ell/2}=z_x\left(+\frac{\ell}{2},y\right)=$$
$$=\frac{G'(+\ell/2)y^2+Q'(+\ell/2)-ay^2\alpha'(+\ell/2)}{2k}=z_2(y)\geq 0$$
$$z_2(y)=-z_1(y)\ \ \forall y\Rightarrow$$
$$\Rightarrow\begin{cases}G'(+\ell/2)+G'(-\ell/2)=a[\alpha'(+\ell/2)+\alpha'(-\ell/2)]\\ Q'(+\ell/2)=-Q'(-\ell/2)\end{cases} \tag{84}$$

Initial conditions for $\alpha(x)$, G(x) and Q(x) and the parameters a and k can be sought in order to meet the requirement that the membrane surface $z = z(x,y)$ is in the interior of the vault thickness everywhere. Note that the set functions satisfy the equations

$$\frac{\partial^2\Psi}{\partial x^2}\frac{\partial^2 z}{\partial y^2}-2\frac{\partial^2\Psi}{\partial x\partial y}\frac{\partial^2 z}{\partial x\partial y}+\frac{\partial^2\Psi}{\partial y^2}\frac{\partial^2 z}{\partial x^2}=-\overline{p}_z(x)$$
$$\frac{\partial^2\Psi}{\partial x^2}\frac{\partial^2\Psi}{\partial y^2}-\left(\frac{\partial^2\Psi}{\partial x\partial y}\right)^2=\rho(y)k\overline{p}_z(x)>0 \tag{85}$$

$$\frac{\partial^2\Psi}{\partial x^2}\frac{\partial^2\Psi_o}{\partial y^2}-2\frac{\partial^2\Psi}{\partial x\partial y}\frac{\partial^2\Psi_o}{\partial x\partial y}+\frac{\partial^2\Psi}{\partial y^2}\frac{\partial^2\Psi_o}{\partial x^2}=$$
$$=-[1-\rho(y)]2k\overline{p}_z(x) \tag{86}$$

with $0 \leq \rho(y) \leq 1$ for any y, i.e. equilibrium and admissibility vs. vertical load are verified.

If one looks at the z-surface resulting from the above positions, one realizes that this can be assumed as the membrane surface for a confined barrel vault (Fig. 26).

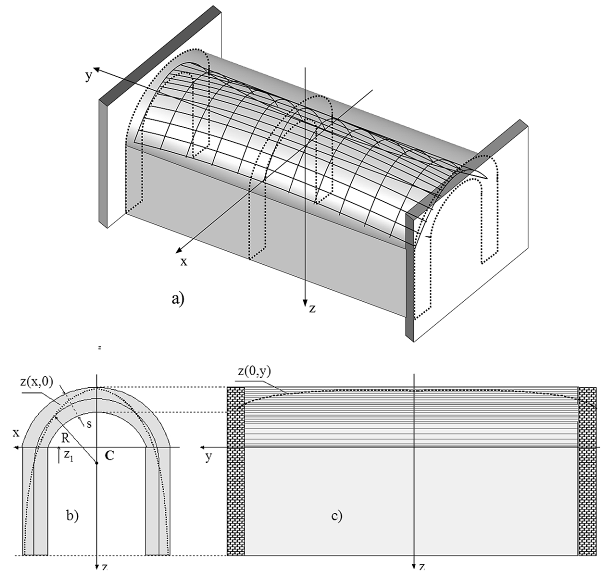


Fig. 26. The confined barrel vault with a possible membrane surface: a) Axonometric projection; b) Transverse xz section; c) Longitudinal yz section

7. VAULTS OF GENERAL SHAPE. THE MONGE-AMPERE EQUATION

The set S of solutions of the *Vault Inequality System (VIS)* composed by the inequalities

$$\begin{cases} \mathcal{H}_z(x,y) = z_{,xx} z_{,yy} - (z_{,xy})^2 \geq 0 \\ z_{,xx}(x,y) \geq 0 \;\; ; \;\; z_{,yy}(x,y) \geq 0 \quad \forall (x,y) \in \mathsf{X} \\ z_1(x,y) \leq z(x,y) \leq z_2(x,y) \end{cases} \tag{87}$$

with X the horizontal projection of the vault, contains all solutions of the z-equilibrium Eq. (58). Solutions of the system Eq. (87) define convex functions $z(x,y)$ included in the profile of the vault, and enjoy the following property: If $z_i(x,y)$ ($i = 1,\ldots,N$) are N functions, each verifying the, possibly homogeneous, basic VIS system in Eq. (87), any convex combination of such functions also yields a member of the solution set of the vault inequality with the strict inequality sign. The homogeneous VIS is the same as Eq. (87) with the sign of equality in the first row.

7.1. Finding solutions

It is possible to build up a number N of solutions $z_i(x,y)$ ($i = 1,\ldots,N$) of the homogeneous or non-homogeneous VIS, so that solutions of the z-equilibrium equation

$$Q \cdot \mathcal{H}_z(x,y) - p_z(x,y) = 0 \tag{88}$$

can be searched in the form

$$z(x,y|c_i) = \sum_{i=1}^{M} c_i z_i(x,y) \tag{89}$$

where each of the basic functions $z_i(x,y)$ is assumed to comply with the VIS [Eq. (87)], in homogeneous or non-homogeneous form.

Assume, moreover, that some c_{oi} coefficients have been found such that

$$z_o(x,y|c_i) = \sum_{i=1}^{M} c_{oi} z_i(x,y) \tag{90}$$

verifies the inclusion condition. To this aim, it is sufficient that Eq. (90) is a convex combination of the basic z_i's, possibly approaching as much as possible the middle surface $z_m(x,y)$.

Coefficients c_i yielding solutions of Eq. (88) correspond to the minimum of the error function

$$\begin{aligned} \mathcal{E}_o(c_i,Q) &= \iint_{\mathsf{X}} [Q\mathcal{H}_z(x,y \mid c_i) - \bar{p}_z(x,y)]^2 \, dxdy = \\ &= Q^2 S(c_i) - 2QR(c_i) + V \end{aligned} \tag{91}$$

with

$$\begin{aligned} S(c_i) &= \iint_{\mathsf{X}} [\mathcal{H}_z(x,y \mid c_i)]^2 \, dxdy \geq 0 \\ R(c_i) &= \iint_{\mathsf{X}} [\mathcal{H}_z(x,y \mid c_i)\bar{p}_z(x,y)] dxdy \\ V &= \iint_{\mathsf{X}} [\bar{p}_z(x,y)]^2 \, dxdy > 0 \end{aligned} \tag{92}$$

One should notice that, since $z(x,y \mid c_i)$ is assumed to be convex, also $R(c_i) \geq 0$ and because of the Schwarz's inequality

$$S(c_i)V - R^2(c_i) \geq 0 \tag{93}$$

The conditions for the minimum of Eq. (91) with respect to Q are written

$$\begin{aligned} \left.\frac{\partial \mathcal{E}_o(c_i,Q)}{\partial Q}\right|_{Q=Q^*} &= 2Q^* S(c_i) - 2R(c_i) = 0 \\ \left.\frac{\partial^2 \mathcal{E}_o(c_i,Q)}{\partial Q^2}\right|_{Q=Q^*} &= 2S(c_i) \geq 0 \end{aligned} \tag{94}$$

whence

$$Q^* = \frac{R(c_i)}{S(c_i)} = \frac{\iint_{\mathsf{X}} [\mathcal{H}_z(x,y \mid c_i)\bar{p}_z(x,y)] dxdy}{\iint_{\mathsf{X}} [\mathcal{H}_z(x,y \mid c_i)]^2 \, dxdy} \geq 0 \tag{95}$$

After substitution in the expression in Eq. (91) of the mean square error, one gets

$$\begin{aligned} \mathcal{E}^*(c_i) &= \mathcal{E}_o(c_i,Q^*) = Q^{*2} S(c_i) - 2Q^* R(c_i) + V = \\ &= V - \frac{R^2(c_i)}{S(c_i)} = \frac{1}{S(c_i)} \left[S(c_i)V - R^2(c_i) \right] \end{aligned} \tag{96}$$

By virtue of Eqs. (92)-(93), $\varepsilon^*(c_i)$ is a positive definite function of its arguments c_i and it is null if $\mathcal{H}_z(x,y) = \bar{p}_z(x,y)$.

It follows that the minimum of $\varepsilon^*(c_i)$ is attained where the ratio $\sim R^2(c_i)/S(c_i)$ is minimum. Therefore the problem can be definitely set in the form

$$\mathcal{E}(c_i) = -\frac{R^2(c_i)}{S(c_i)}$$

$$\mathcal{E}(c_i^*) = \mathcal{E}^* = \min_{c_i} -\frac{R^2(c_i)}{S(c_i)}$$

$$sub$$

$$z_1(x, y) \le z(x, y|c_i) \le z_2(x, y) \quad \forall (x, y) \in \mathsf{X} \qquad (97)$$

All load patterns $\bar{p}_z\,(x,y)$ such that coefficients c_i resulting in $\varepsilon^*=0$ exist, are called *manageable load patterns* with respect to the assumed form for the function $z(x,y\,|\,c_i)$. If the applied load is manageable, equilibrium can be exactly satisfied. Otherwise, equilibrium can be approximately verified, to some extent, depending on the choice of the basic functions $z_i(x,y)$.

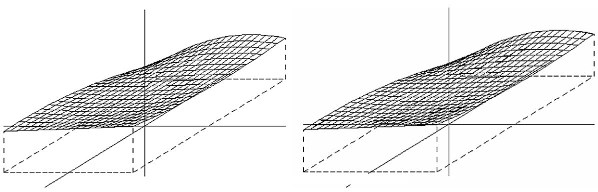
Further details and applications are illustrated in [17, 18]. Interaction with the reciprocal problem, i.e. No-Compression double curvature structures, can give fruitful contributes to new developments (see e.g. [19]).

7.2. Some results. The NT vault subject to the self-weight and to a manageable live load

As an example, a parabolic vault covering a rectangular domain $\mathsf{X} = [x_1,x_2] \times [y_1,y_2]$. The vault is characterized by height f, thickness s and middle surface

$$\bar{z}_m(x, y) = a\left(x^2 + y^2\right) + s \qquad (98)$$

with $a = f/(x_1^2 + y_1^2)$.



a) b)

Fig. 27. Sample results: a) Plot of the objective load p(x,y) = w(x,y)+ q(x,y); b) Superposition of the objective load and its approximation by means of $\mathcal{H}z(x,y)$; the two plots are practically coincident

The objective load is composed by the superposition of an accidental load $q(x,y)$ localized around a point (x_o, y_o) and the self weight of the vault $w(x,y)$ given as follows

$$w(x, y) = \gamma_f\left[\bar{z}_1(x, y) + t\right] + \gamma_m s \qquad (99)$$

with t denoting the height of the superstructure, γ_f and γ_m the unit weight of the superstructure and of the structural masonry respectively.

A manageable live load is assumed in the form

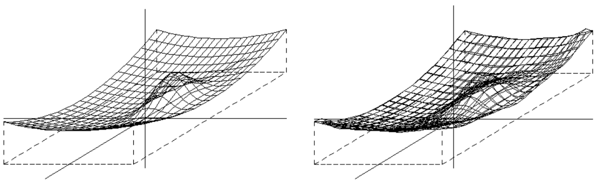
$$q(x, y) = q_o\left[(x_1 - x_o)^2(y_1 - y_o)^2 - (x - x_o)^2(y - y_o)^2\right] \qquad (100)$$

and the results are plotted in Fig. 27, proving a perfect agreement between the Hessian $\mathcal{H}_z(x,y)$ of the function $z(x,y)$ and the objective load $p(x,y) = w(x,y) + q(x,y)$.

A second example is quoted in Fig. 28, where a live load, non-manageable with respect to the basis adopted for $z(x,y)$ is assumed in the form

$$q(x, y) = q_o \exp\left\{-\left[\frac{(x - x_o)^2}{2\Delta_x^2} + \frac{(y - y_o)^2}{2\Delta_y^2}\right]\right\} \qquad (101)$$

The results are not coincident, but anyway a good approximation can be obtained. Better results can be pursued, of course, assuming an expression for $z(x,y)$ that makes the load manageable. Results are synthetically quoted in Fig. 28.



a) b)

Fig. 28. Non-manageable load pattern: a) Plot of the objective load p(x,y) = w(x,y)+ q(x,y); b) Superposition of the objective load and its approximation by means of $\mathcal{H}z(x,y)$

8. CONCLUSIONS

Historical masonry vaults and/or cupolas exhibit a large variety of typological assets. Often masonry is well operated, with strong stones and effectively adhesive mortar; in many cases masonry is in worse working order; in other cases a poor masonry is encountered.

Anyway, double-curvature structures can appeal to many equilibrium patterns to sustain at least their own weight plus some light additional loads. So they are in general stable systems, provided that their supports are strong and able to contrast thrust forces. Vaults are in general characterized by their shape, and a lot of types can be listed (see e.g. [20]), that have been conceived to be included in any simple or complex architectural design. But the equilibrium paths are also driven by the way masonry is interwoven. In some cases, a masterly design of the masonry tissue and of the vault apparatus may help in improving the structure's stability, and sometimes even in preventing fractures, as discussed and illustrated in Sec. 3.

It should be realized, by contrast, that fractures are almost always a physiological feature of masonry; since almost always it does not enjoy significant tensile strength, it cannot expand by tension and, when necessary to comply with congruence of the overall deformation, dilatation is provided by fractures.

Anyway, the poor consistency of the tensile resistance of the masonry material, its brittle, desultory and time-aging character, the difficulty in identifying non-zero reliable values, possibly led the ancient builders to introject empirical rules aiming, more or less consciously, at organizing structures in way that they are able to equilibrate loads without needing tensile stresses in the material. A susceptibility that has not been disproved by any analysis performed by modern powerful theoretical, numerical and electronic equipments. After a similar survey, Heyman, in 1966 [21], demonstrated that the failure of the masonry structures was substantially due to the activation of a collapse mechanism, rather than to the probability of compression crushing.

Many efforts have been devoted to specializing the Limit Analysis theory to the case of masonry structures (see e.g. [22-27]). Actually by referring to the collapse condition, one can just obtain some indications about the safety margins, whilst nothing about the fracture distribution or the behaviour evolution with increasing loads can be predicted.

Under such perspective, even if the NT model still represents an idealization of the real behaviour, one can follow the fracture evolution, assuming the small localized fractures as a phenomenological feature of the masonry material, besides the cases when the crack situation is such to compromise the local material resistance or to activate a collapse mechanism. The special character of the model, which results in a overall non-linear behaviour due to the unilateral nature of the constitutive law, and in a few more features related to the basic assumptions (determining and governing the existence of the inelastic crack strain field, the conditions for its activation, its development at the first detachment, the characteristics of the stress state), require some special formulation for structural analysis purposes (Sec.4).

Definitely, the solution of the structural problem is based on a suitable re-formulation of the energetic theorems, which, by reflecting the non-linear character of the mechanical model, translates into constrained extremum principles the ordinary conditions of stress equilibrium on one side, and of strain congruence on the other side. In such a way, the final state of the structural solid under different load levels can be identified up to the collapse situation, which can be predicted, as mentioned in the above, by means of the fundamental theorems of Limit Analysis, suitably re-formulated. As far as two-dimensional structural systems are concerned (walls, arches, plane models of masonry bridges and so on), and some their combinations, theory and practice are in a well established state of the art.

Application to the statics of vaults and cupolas is today largely investigated by many authors. The present paper is far from aiming at an exhaustive review, and only a few papers (see e.g. [28-33]) are referenced here, among the manifold that would deserve to be mentioned. It is clear however that the problem is much harder than for plane structures; in this regard it may be enough to consider the intrinsic difficulties to identify collapse mechanisms in applying the kinematic approach of Limit Analysis to double-curvature vaults.

A common feature, however, is that in most cases the main objective is to extend to double-curvature roofing the methods that have historically developed with reference to single curvature arches or analogues, to find admissible stress distributions. The approach illustrated in Secs. 5-7, originally elaborated by the writers, aims at this purpose, on one side providing solution to the collapse problem from the point of view of the static theorem, and on the other side solving the preliminary step to find solutions including compatible strain and fracture fields in agreement with field engineering surveys. The Monge-Ampere equation, introduced in Sec. 5, is essentially the double-curvature counterpart of the equation of the funicular line. The equation has widely been investigated by mathematicians; nevertheless only a few solutions are available in the literature, so that solutions effective for the problem at hand shall be sought by specific and/or numerical methods. A Ritz-Galerkin type approach, requiring a previous identification of a number of basic functions, has been specifically issued in Sec. 7, proving its manageability in practical examples.

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Abstract

After discussing the problem of roofing empty spaces by ancient masonry builders, it is found out that curvature and horizontal thrust are the basic elements for masonry to get over long spans. Basic properties of masonry do not allow to rely on tensile strength, and beam behaviour cannot be trusted on. Nevertheless in 2D walls and in double curvature vaults, a particular organization of the vault apparatus can in some instances, through the action of compression and friction, give place to a equilibrium pattern including tension, which explains the unexpected good performance of some walls and cupolas. Anyway, it is recognized that, apart from a few cases, the No-Tension assumption yields a effective model for structural assessment. The theory is briefly illustrated, and its application to vaults is explained in detail, leading to a Monge-Ampere equation ruling the static regime through a membrane stress surface.

Streszczenie

Po przeanalizowaniu problematyki przekryć wznoszonych techniką murarską w dawnych czasach, okazało się, że krzywizna i siły rozporu są głównymi elementami pozwalającymi wykonać murowane konstrukcje ponad dużymi przestrzeniami. Podstawowe właściwości budulca nie gwarantują wytrzymałości na rozciąganie, ani nie dają pewności w kwestii pracy belek. Jednakże w ścianach 2D i sklepieniach o podwójnej krzywiźnie, specyficzna konfiguracja sklepienia może w niektórych przypadkach, dzięki działaniu sił ściskających i tarcia, pozwolić na powstanie modelu równowagi obejmującego rozciąganie, co wyjaśnia nadspodziewaną nośność niektórych ścian i kopuł. Ogólnie uznaje się, że z wyjątkiem nielicznych przypadków, założenie braku naprężeń rozciągających daje w efekcie odpowiedni model do oceny konstrukcji. Ta krótko przedstawiona tutaj teoria i jej zastosowanie w przypadku kopuł, zostały szczegółowo wyjaśnione, czego efektem jest równanie Monge-Ampere wyznaczające schemat statyczny w naprężeniu błonowym powierzchni.

Jacek Kościuk[1]

Modern 3D scanning in modelling, documentation and conservation of architectural heritage

Współczesne skanowanie laserowe 3D w modelowaniu, dokumentacji i konserwacji zabytków architektury

Keywords: Analysis, Construction, Architectural heritage, Documentation, TLS

Słowa kluczowe: Konstrukcja, zabytki architektury, konserwacja, dokumentacja, laserowe skanowanie 3D

1. INTRODUCTION

Twenty two years ago, in 1990, Ben Kacyra, Iraqi expatriate and civil engineer, launched in USA the world's first 3D commercial laser scanner. Since that time, the new technology – terrestrial laser scanning (TLS) has been gaining momentum, both advancing technologically and ensuring broader and broader recognition as a reliable measuring instrument in many disciplines. Not surpassingly, it has also found its way into civil and structural engineering, as well as into documentation in the field of conservation of architectural heritage. It is enough to type only *Laser scanner and heritage* phrase into Google Scholar to see more than six thousand papers published on this subject since 1990. Typing *Laser scanner and structural applications* phrase will end with an even higher number – nearly 16 thousand search results for the same period (Fig. 1). It is necessary to note, that due to the simplicity of such query, some of these references might be irrelevant to the main subject. Nevertheless, more than 5 thousand publications on this subject expected in the year 2012 alone clearly show that TLS is already recognised as a well established and trusted technology. What is perhaps equally interesting is the fact that the number of related publications shows yearly exponential growth.

By now most structural engineers and specialists in conservation of architectural heritage are not interested if, but rather what for and how as far as TLS is concerned. Different authors show results of successful TLS application in more and more sophisticated cases [1], some others are working on the theoretical background of TLS usage or its measurements accuracy and repeatability [2], still others concern themselves with attempts to establish a good code of practice and TLS applicability standards in different disciplines [3], some are interested in using TLS for detecting displacement and deformations [4]. Since it is virtually impossible to encompass such a broad scope of interest in a limited frame of this keynote paper, I will concentrate on main differences in standards and good code of practice as required from the point of view of three particular scanning aims – structural analyses, architectural design and visualization – all of them mostly in the context of heritage conservation. As the main discriminant, the accuracy of final deliverables and the loss of original data fidelity in the process of data elaboration will be analyzed. Therefore the main leading idea of this paper can be paraphrased as: *documentation or visualization*.

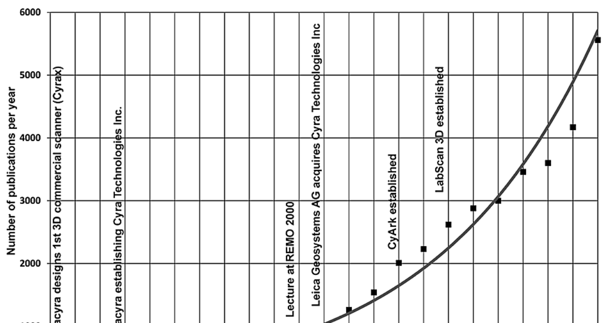

Fig. 1. Results of Google Scholar search on Laser scanner and structural applications phrase (access on 03.05. 2012). Some events in TLS and LabScan3D history are additionally marked

However, from this point of view, one more general question should be answered at the beginning – namely, what is the place of TLS among other modern measuring technologies? Terrestrial laser scanning can be positioned between two other

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surveying technologies: close range digital photogrammetry and kinematic scanning (Fig. 2). All surveying technologies shown in Fig. 2 are characterized by three factors: the size of measured object ranging from a few centimetres to hundreds of kilometres; the density of probing (from friction of a millimetre to several meters); and accuracy (from under a millimetre to several decimetres). Obviously, not all surveying technologies shown in Fig. 2 are in the scope of our interest.

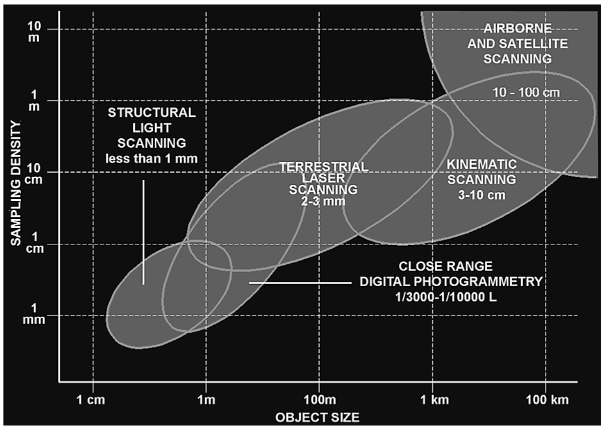

Fig. 2. Modern surveying technologies comparison. Adopted from [3]

Another important factor, which we should take into consideration, particularly from the point of view of the measurement accuracy demanded in most of structural analysis, is the precision of TLS confronted with the desirable precision for this sort of application, as well as with the precision offered by analogous surveying technologies (Fig. 3).

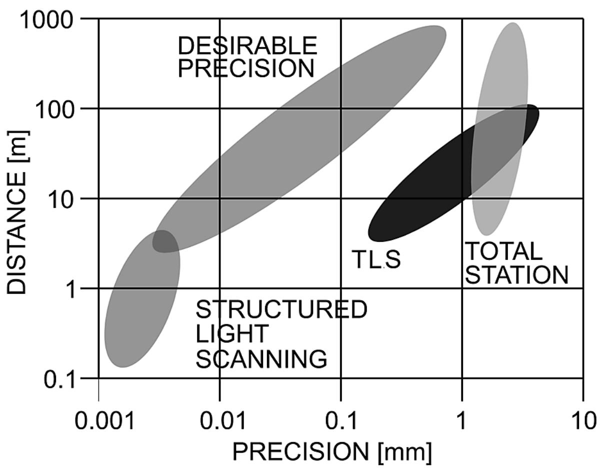

Fig. 3. Precision of some surveying technologies in relation to the measuring distance. Adopted from [5]

Theoretically, the parameter which will satisfy even the most demanding applications can be described as a proportion of the distance from which measurements are recorded to the achieved precision. This desirable proportion falls in a region of 10^{-6} part of the distance and preferably should be constant. However, the existing technologies do not provide us with a single method which will keep such a precision across different scales (Fig. 3). The most advanced methods of structured light scanning can offer even higher precision, but it is practically impossible to maintain this highest parameter above the measuring distance of a few meters. In turn, in case of classical surveying methods utilizing total stations, this proportion will change, ranging from nearly 10^{-3} part for distances around few meters up to ca. 10^{-5} part for distances of several hundred meters. TLS technology shows still different characteristics. The parameter in focus falls in range of 10^{-4} part of the measuring distance and seems to be relatively constant within the range starting from few a meters and ending up at around 100 meters, which is usually the limit for applications of our interest.

2. TLS DELIVERABLES IN RELATION TO THEIR ACCURACY AND LOSS OF ORIGINAL DATA FIDELITY IN THE PROCESS OF DATA ELABORATION

The most important factor from our main point of view, namely the suitability of TLS in documentation and conservation of architectural heritage with special respect to structural analysis, is accuracy and loss of original data fidelity in the process of preparing final deliverables. We can state that accuracy and loss of original data fidelity is the main discriminant which separates *documentation* from *visualization*. Here the last two terms are used to distinguish between deliverables which meet accuracy standards required in certain fields of application from these which are merely providing us with a pictorial illustration of the studied object. For example, *documentation* for museum purposes, for architectural design or structural analyzes, deterioration studies, displacement and deformation monitoring, etc., each will require their own accuracy, precision and data density, as opposed to *visualization* meant only to illustrate or describe the studied object in a broad sense. Unfortunately, these distinctions between credible *documentation* and often very impressive *visualization* is in many cases not fully recognized. This situation calls for establishing certain standards in using TLS as a method for documentation and conservation of architectural heritage or in structural analyses. Despite many attempts of different authors [1], [6], it is difficult or even impossible to come across a comprehensive and fully satisfying approach for such standardization. Neither does this humble lecture aim to solve this issue.

In table 1 represented below, the author tries to classify main types of TLS deliverables in relation to their accuracy and loss of original data fidelity in the process of data elaboration. The lower position in the table, the inferior the accuracy and data fidelity.

As can be seen from the table 1, viewing (visualizing) 3D point cloud (Fig. 4) in its 3D digital environment does not affect original data accuracy and fidelity. Depending on hardware and workflow used, the recorded 3D point cloud can include or not, information about RGB colours for each 3D point. When no RGB values are recorded we are only offered pure geometric information (X, Y, Z coordinates of each 3D point) supplied with the value of reflection intensity. The last can be represented with a colour palette of any choice, including grey scale palette which to some degree resembles black and white photo representation. In turn, RGB values information can be acquired either directly from sensors built into hardware equipment or ported from digital imagery and added in post-processing. However, in both cases we must expect certain degree of inaccuracy. Much smaller in case of colour sensors built directly into hardware equipment and rectified by the manufacturer – in this case the parallax between X, Y, Z geometry and the colour information is usually negligible. However, the quality of colour and its resolution plays also very important role. In the case of colour sensors built directly into hardware equipment, the quality of colour tends to be inferior, while image resolution is usually higher. When adding RGB information in post-processing mode, the risk of discrepancy between X, Y, Z geometry and colour the information is much higher. It depends on several factors – the quality of digital camera used, quality of software used to merge colour information with 3D point data, and obviously skills and experience of the software operator.

Table 1. Main types of TLS deliverables in relation to their accuracy and lose of original data fidelity in process of data elaboration

3D data as recorded by TLS – 3D point cloud	
2D representation	3D representation
black & white (intensity scale) orthophoto delivered directly from 3D point clouds	viewing (visualizing) 3D point cloud in reflection intensity mode
colour (RGB) orthophoto delivered directly from 3D point clouds and colour photomosaics* calibrated with 3D scan data	viewing (visualizing) 3D point cloud in colour (RGB) mode
2D line drawings (plans, views, sections) delivered manually or semiautomatically directly from 3D point clouds	3D line wireframe drawings (plans, views, sections) delivered manually or semiautomatically directly from 3D point clouds
orthophoto delivered from mesh models textured with black & white or colour information	3D mesh models delivered from 3D point clouds
2D line drawings (plans, views, sections) delivered from 3D mesh models	
2D line drawings (plans, views, sections) delivered manually or semiautomatically from 3D surface models	3D surface models delivered manually or semiautomatically directly from 3D point clouds
2D line drawings (plans, views, sections) delivered automatically from 3D solid models	3D solid models delivered manually or semiautomatically directly from 3D point clouds (BIM models)
2D line drawings (plans, views, sections) delivered by manual or semiautomatic on-screen digitizing of orthophotos or photomosaics	–

** in fact, colour photomosaics calibrated with 3D scan data only roughly and in some particular conditions meet orthophoto accuracy. However, they bring much better textural information.*

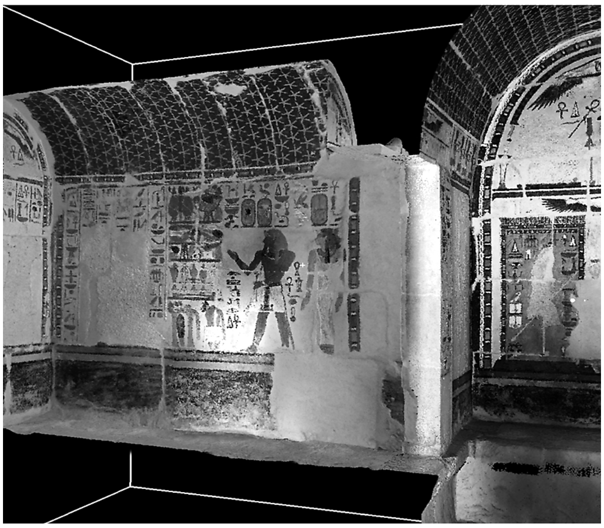

Fig. 4. Example of viewing (visualizing) 3D point cloud in colour (RGB) mode. Upper Anubis Shrine in Hatshepsut Temple in Deir el Bahari (Egypt)

One of the side problems, which should be again mentioned, is lack of standards. There is no worldwide recognized standard for 3D laser scanning for world heritage documentation, or for any other particular fields of 3D laser scanning application including structural analyses. This situation often results in an approach *let scan as dense as possible in given circumstances*, which for many reasons does not always seem to be the best choice, if at all. The situation is made even more complex by lack of common standards of 3D point cloud data formats. Each hardware manufacturer develops their own proprietary format, with capabilities of interoperability which often prove limited. There is currently no general-purpose, open standard for storing point-cloud data, yet there is a critical need in the 3D-imaging industry for open standards that promote data interoperability among hardware and software systems [7]. Among other initiatives, recent activities of SPIE (an international society advancing an interdisciplinary approach to the science and application of light), particularly works on establishing ASTM E57 file format are likely to change this situation [8].

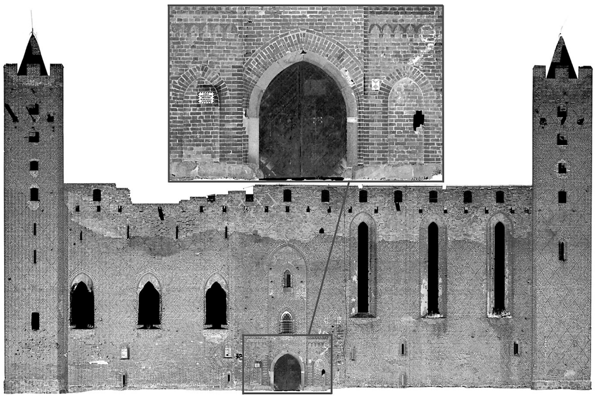

Fig. 5. Example of black & white (intensity scale) orthophoto acquired directly from 3D point cloud. Eastern façade of mediaeval Teutonic Knights castle in Radzyń (Poland)

Nevertheless, storing original 3D cloud point databases may be considered as the best way to document objects for archival purposes. Such the data deposited in safe repositories can be later consulted at any time and in any case. Unfortunately, usefulness of 3D point clouds in its original form as a direct data for design or structural analyses, to mention only these two possible fields of application, is very limited. The most recent attempts to immerse 3D design files directly into 3D point clouds representations might solve this problem in a near future. At least, all leading CAD vendors seem to work hard on this issue.

There is, however, a big discrepancy between current software's abilities to handle 3D point cloud data and the way typical designers and engineers used to work. Still most of us tend to work rather in 2D data format. How can this group enjoy benefits of TLS technology? The only answer is to work with 2D representation of 3D point clouds. Among many types of 2D TLS deliverables, black & white (intensity scale) orthophoto acquired directly from 3D point clouds is the first answer if we still consider data accuracy and loss of original data fidelity in process of data elaboration as an important factor (Fig. 5). High accuracy data with resolution of around 2-3 mm can be directly inserted into any CAD application assuring good representation of documented object.

A still inferior choice are, again from the point of view of data accuracy, colour (RGB) orthophotos delivered directly from 3D point clouds. Depending on the way how the colour information has been ported into 3D point cloud we may expect smaller or bigger accuracy issues. However, additional information obtained from colour representation has high value by itself (Fig. 6). Both kinds of raster orthophoto files can be inserted directly into any CAD application as a background for design or any analyses. It is important to stress that there is no particular necessity to redraw this raster information in vector format. To the contrary, such an action will affect data introducing interpretation and simplification errors.

Fig. 6. Example of colour (RGB) orthophoto acquired directly from 3D point cloud. Southern façade of mediaeval bishop palace in Milicz (Poland)

Fig. 7. Example of displacement map delivered directly from 3D point cloud. Southern wall of Upper Anubis Shrine in Hatshepsut Temple in Deir el Bahari (Egypt)

The possibility to represent 3D cloud points in any arbitrarily chosen colours opens another field of application. When turning the coordinates system in such a way that Z axis is perpendicular to the surface of our interest, we can represent all 3D points in different colours according to their Z value (the so called elevation maps). This is perhaps the fastest, the most simple and at the same time still very accurate way to represent defects in any planarity and can be successfully used for example in wall displacement detection (Fig. 7).

There are however situations when 2-3 mm resolution offered by orthophotos acquired directly from 3D point cloud is fairly not satisfactory and much higher data resolution is demanded. A typical example are documentation works on wall paintings or bas-relief. In such a situation, particularly when the depth of the object (a relief for example) is negligible if compared to the distance from which the photography is taken, we can use colour photomosaics calibrated with 3D scan data (Fig. 8). Thus however, we are trading geometrical data accuracy for higher resolution. If we accept the resulting inferior data accuracy, resolutions of 0,3 mm are relatively easy to obtain [9].

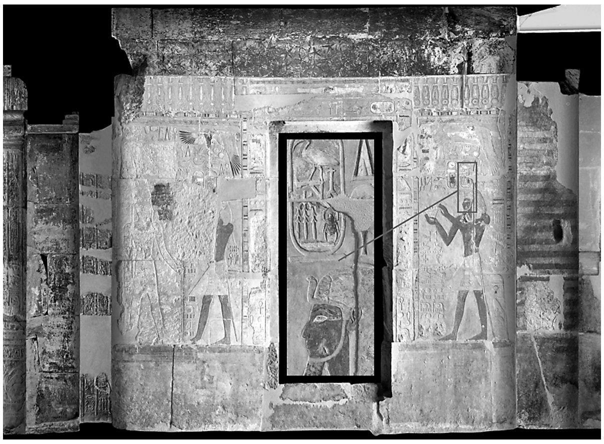

Fig. 8. Example of 0,3 mm resolution photomosaic calibrated with 3D scan data. Northern wall of Statue Room in the Main Sanctuary of Amun in Hatshepsut Temple in Deir el Bahari (Egypt)

Going back to the direct use of 3D data (Table 1) we should mention 3D line wireframe drawings (plans, views, sections) delivered directly from 3D point clouds. Most of software applications meant do deal with 3D point cloud data have built in algorithms to extract such data in manual, semiautomatic or even fully automatic mode. Aside from the data simplification, the resulting wireframe 3D models are of limited use both in documentation and design, as well as in structural analyses. 2D representation of such models, which still inherits all the handicaps of data simplification, can be, according to my experience, used in a bit broader range of applications.

Next in Table 1 comes the 3D mesh models delivered from 3D point clouds. Obviously, implemented meshing algorithms introduce some unavoidable interference into original data. Depending on applied the meshing parameters this could be data density decimation, smoothing, filtering, hole detection and surface reconstruction, etc. Thoughtfully used, all these algorithms are likely to improve obtained results, but the degree of interference into original data should be kept in mind all the time. Generally speaking, 3D mesh models show high potential for representing documented surfaces with good fidelity, but at the cost of large size of data files. The more detailed mesh file, the bigger the file size. Mostly for this reason, the usage of 3D mesh files directly in 3D CAD applications is rather limited. However, mesh files offer a good starting point for many kinds of different deliverables ranging from orthophotos textured with black & white or colour information which can be also produced from mesh models (Fig. 9), up to 2D or 3D line drawings (plans, views, sections) obtainable from mesh models often just by 1 mouse click (Fig. 10). Mesh models also constitute valuable basic data to enter the world of finite element method (FEM) analyzes. Additionally, mesh models can be successfully used in deformation analyzes. Scanning the object of our interest in certain time intervals and then representing its surfaces as mesh models permits detection of any changes caused by many different factors. This method has recently been successfully and broadly used in landslides and constructions monitoring (Fig. 11).

Fig. 9. Example of orthophoto delivered from 3D mesh model with some overlaying 2D vector lines. Mediaeval chapter-house in Wrocław (Poland)

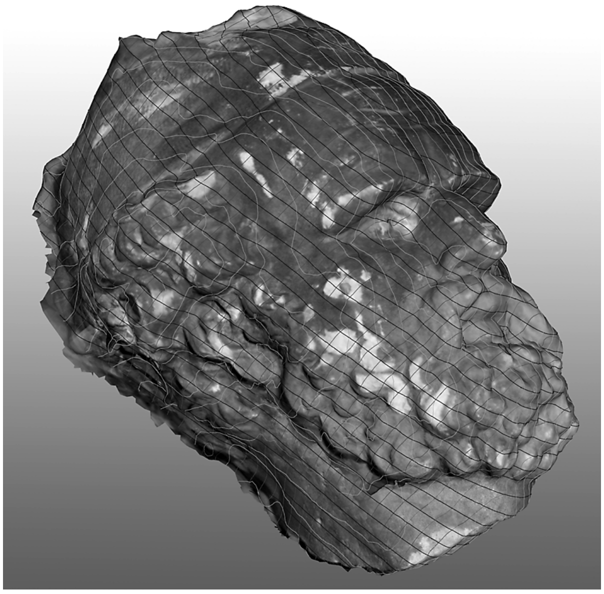

Fig. 10. Example of 3D mesh model with overlaying 3D vector lines. Head of a statue of unknown provenance (deposit of Institute of History of Architecture, Arts and Technology, Faculty of Architecture, Wroclaw University of Technology)

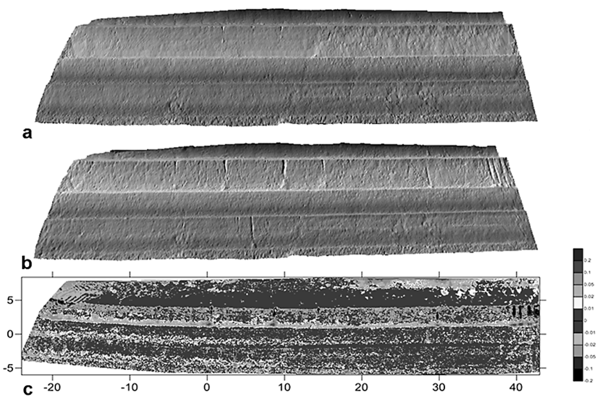

Fig. 11. Example of deformation analyses based on 3D mesh models. a – 3D mesh model of primary stage of ground slope, b – 3D mesh model of the same ground slope after erosion, c – results of 3D mesh models subtracting shows amount of ground slope displacement. Adopted from [10]

The next step of simplification of real 3D world representation are 3D surface models delivered manually or semiautomatically directly from 3D point clouds (Table 1). In this case, the degree of simplification is much higher but the resulting files are at the same time much smaller in size, and therefore easier to handle by CAD applications. As before, a similar kind of 3D and 2D deliverables can be achieved out of such models. Although the usage of 3D surface models in CAD application is much broader, it still does not fully meet possibilities offered by the functionality of contemporary design software. This stage can be fully implemented only with solid modeling which is the entry point for building information modeling software (BIM). There all the objects are represented as much simplified true solids (Fig. 12) which are easy to handle by CAD algorithms. At the same time we lose most of the detailed information about the documented objects. All the walls used to be represented as perfectly planar, while beams, posts etc. loose all the information about their imperfectness (cracks, deflection, twisting, etc.).

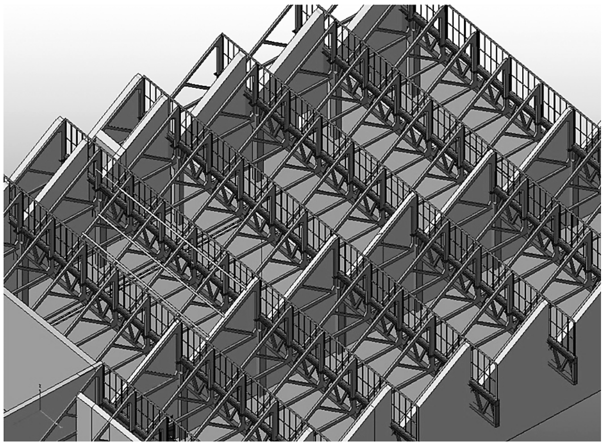

Fig. 12. Example of 3D solid model delivered directly from 3D point cloud (BIM model). Prefab concrete industrial building in Wrocław (Poland)

Despite the loss of so much detailed information about the geometry of the documented object, there is however one big advantage. Solid models can be easily incorporated into the abovementioned BIM software and, when supplied with additional data-based information (material, weight, physical and mechanical properties, etc) constitute a contemporary high-end standard for design as well as analytical and struc-

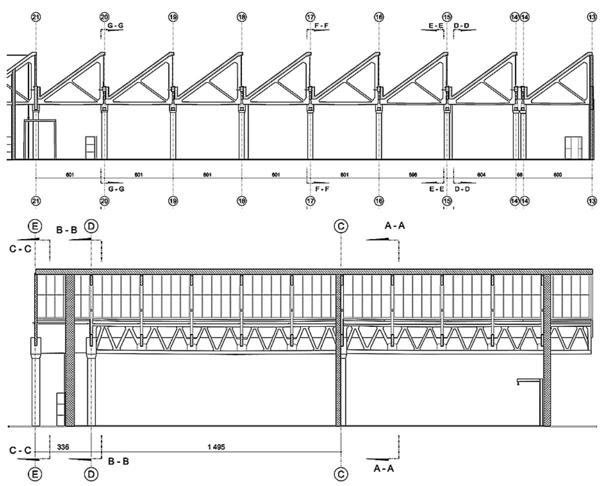

Fig. 13. 2D vector drawings delivered automatically from the BIM model represented on Fig. 12

tural applications. Such models are equipped with a kind of their own intelligence – all elements are not only described in terms of spatial geometry, but they also *know* how to present (visualize) themselves in perspective views, plans and sections, so automatic production of 2D documentation is fully possible (Fig. 13). Solid models are also best suited for structural FEM analyses (Fig. 14).

There is still one category of 2D deliverables which has been omitted from a more detailed discussion – 2D line drawings (Fig. 14.) delivered by manual or semiautomatic on-screen digitizing of orthophotos or photomosaics, which in turn are products of 3D laser scanning. There we must recall all what has been previously said about the production of orthophotos or photomosaics. With some reservations concerning the accuracy of RGB colour mapping, orthophotos produced directly from 3D point cloud can be considered a very accurate 2D representation of 3D world. What happens when such the data undergoes on-screen digitizing?

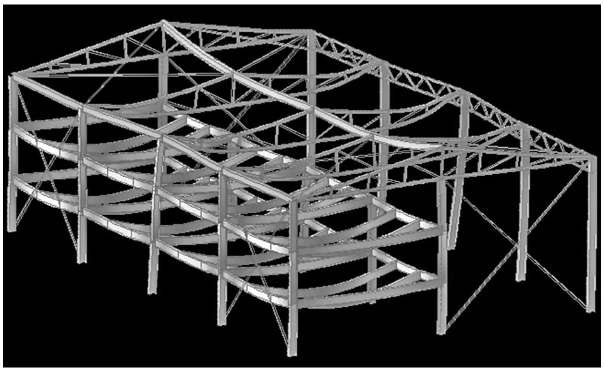

Fig. 14. Example of solid model of steel construction during FEM analyses

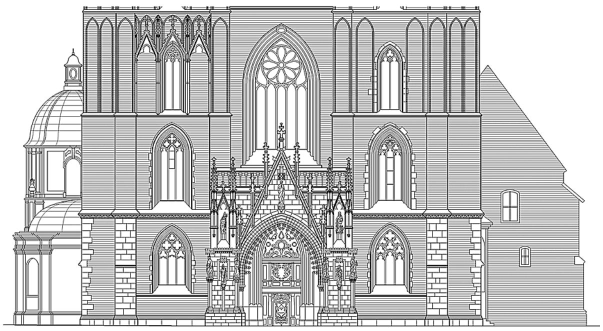

Fig. 15 . Fragment of western façade of Wroclaw Cathedral. 2D line vector drawing delivered from on-screen digitizing. © Archikon s.c.

On-screen digitizing of raster data is commonly used in cartography, where specially tailored software applications are used in order to assure demanded accuracy. However, when using such software for digitizing orthophotos produced directly from 3D point cloud we encounter some problems. In this case, raster data we deal with are of a slightly different character. Neither are they full halftone colour pictures as it is in case of airborne imagery, nor sharp, limited colour palette images as typical cartographic maps. This results in certain difficulties in proper usage of algorithms designed to trace certain colour differences yielding with decreasing accuracy of vectorized data. Situation becomes even worse with digitizing orthophotos delivered from mesh models. As it was previously pointed out, such orthophotos are inheriting all the faults of the meshing process, so the final degree of accuracy constitutes the sum of errors from the two steps: meshing and digitizing. There is of course no problem as long as we are aware of it and as long as final accuracy meets our demands.

However, in daily practice I have observed so often, on-screen digitizing done manually without help of any dedicated software. Can we consider such the data as fully credible 2D documentation – deliverables of high-end TLS technology – or do they rather fall into *visualization* category which only merely illustrates the real world? Fully arbitrarily chosen vectorization criteria, no control upon the resulting accuracy – all this makes me perceive such data fall as belonging rather to the *visualization* category. They might still be useful in some cases, but it is rather hard to consider them true documentation. Their usage in structural analyses is also highly problematic.

3. CONCLUSIONS

When trying to summarize what has been previously said, we may end up with something similar to the Table 2 represented below.

Table 2. Applicability of different TLS deliverables according to their suitability for certain purposes

Field of application / Types of deliverables	archival documentation*	architectural design	structural design	structural analyses	deterioration analyses	deformation and displacement monitoring
black & white (intensity scale) orthophoto delivered directly from 3D point clouds	+/-	+	--	---	+	+
colour (RGB) orthophoto delivered directly from 3D point clouds	++	++	--	---	+	++
colour photomosaics* calibrated with 3D scan data	++	++	---	---	++	---
2D line drawings (plans, views, sections) delivered manually or semiautomatically directly from 3D point clouds	+	+/-**	+**	--	+**	+**
orthophoto delivered from mesh models textured with black & white or colour information	+	+	--	-	+	++
2D line drawings (plans, views, sections) delivered from 3D mesh models	+/-	++	+	++	+	++
2D line drawings (plans, views, sections) delivered manually or semiautomatically from 3D surface models	-	++	++	+	-	+
2D line drawings (plans, views, sections) delivered automatically from 3D solid models	--	+++	++	+	+/-	+/-
2D line drawings (plans, views, sections) delivered by manual or semiautomatic on-screen digitizing of orthophotos	---	+	--	---	--	---
original 3D point cloud	+++	+/-	+/-	+/-	+/-	+
viewing (visualizing) 3D point cloud in reflection intensity mode	---	---	---	---	---	+/-
viewing (visualizing) 3D point cloud in colour (RGB) mode	--	--	---	---	---	+
3D line wireframe drawings (plans, views, sections) delivered manually or semiautomatically directly from 3D point clouds	-	---	--	+	---	+**
3D mesh models delivered from 3D point clouds	+	--- ***	---	++	+++	+++
3D surface models delivered manually or semiautomatically directly from 3D point clouds	--	--***	--	++	--	+
3D solid (BIM) models delivered manually or semiautomatically directly from 3D point clouds (BIM models)	---	+++	+++	+++	---	-

note: number of +/ – signs describe suitability of certain deliverables for different purposes.

* *archival documentation is meant there as a possibly full and exact geometrical and textural representation of a given object.*

** *high accuracy but time-consuming*

*** *too large files and too detailed representation for most of architectural software*

The classification presented above is fairly subjective and results from author's merely twelve years of theoretical and practical experience with TLS technology, rather than from particular scientific research and as such can be questionable. Despite possible criticism and different points of views resulting from different practical experiences or research, I suggest that problems presented in this paper should be always kept in mind, both by those using TLS technology and by the end clients expecting to get credible results, which will meet their requirements concerning accuracy and data fidelity.

ACKNOWLEDGEMENTS

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Abstract

The paper deals with broad field of terrestrial 3D laser scanning (TLS) data application in documentation and conservation of architectural heritage. Position of TLS among other surveying technologies is shown in respect for density of information, size of surveyed objects and expected accuracy. Different kinds of 2D and 3D deliverables are characterized with the main focus on their accuracy and loss of original data fidelity in process of data elaboration. Finally, the problem of applicability of different TLS deliverables is discussed.

Streszczenie

Artykuł dotyczy szeroko rozumianych zastosowań naziemnego skanowania laserowego (TLS) w dokumentacji i konserwacji zabytków architektury. Omówiono miejsce technologii TLS wśród innych technik pomiarowych z uwzględnieniem gęstości próbkowania, wielkości mierzonych obiektów i oczekiwanych dokładności. Scharakteryzowano także podstawowe rodzaje opracowań 2D i 3D otrzymywanych w wyniku laserowego skanowania w odniesieniu do uzyskiwanych dokładności i utraty części pierwotnej informacji w procesie opracowania danych. W zakończeniu przedstawiono potencjalny zakres zastosowań dla poszczególnych rodzajów opracowań uzyskiwanych w wyniku naziemnego, laserowego skanowania 3D.

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Structural analysis of complex forms in the german baroque

Analiza konstrukcji złożonych form z okresu niemieckiego baroku

Keywords: Analysis, Masonry shells, German Baroque

Słowa kluczowe: analiza, powłoki konstrukcyjne, niemiecki barok

1. INTRODUCTION

The research about structural behaviour of brick and stone masonry roofs has been very intensive by the Scientific Community during the last decades. The knowing level of roofs based on canonic forms (cylinders, spheres), like barrel vaults or spherical domes, is very high. In this sense, our research is focused on the structural analysis of complex masonry roofs, in which the creation of the roof surface is not so simple and where another structural schemes appear. In this analysis, we will treat roofs of the Central European Baroque, especially some complex roofsmainly developed by two great architects who were able to built light and geometrically complex masonry roofs made from two different structural schemes.

Roofs designed by Dientzenhofer Dynasty, characterized by a design based on a structural load bearing skeleton, contrast to those designed by Balthasar Neumann, who defend a continuous roof without any kind of structural reinforcement.

2. THE WORK BY DIENTZENHOFER FAMILY

Dientzenhofer Family excels in the design of special compositions of roofs that rest on warped brick masonry arches which are used as structural skeletons. We can appreciate these warped ribs at St. Klara, in Eger (Cheb) (Fig. 1), at St. María Oboriste, in Prague (Fig. 2), or in the east of Germany [1].

Christoph Dientzenhofer was the creator of the warped rib and he uses it in the majority of his works. This rib is defined by the intersection between two cylinders with different diameters. The lack of knowing about intersections between quadric surfaces caused that Christoph Dientzenhofer had to define his roofs from the known intersections between the cylinders that we have just commented (the warped ribs) and the outline, creating compatible surfaces that rest on them.

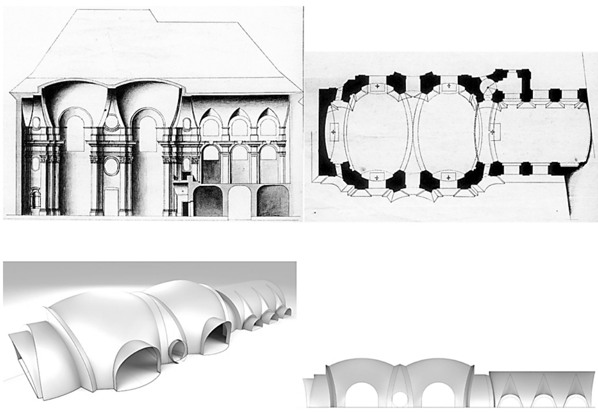

Fig. 1. St. Klara, in Eger. Christoph Dientzenhofer. Ground plan, Section and 3D-model

The first example that we are going to present is the Benedictine Monastery of Banz, the greatest building built by the Dientzenhofer Dynasty, with Johann Dientzenhofer as the main author. On the other hand, we will also present the most representative works made by Balthasar Neumann. He based the design of his brick masonry roofs on the geometrical compositions of the Dientzenhofer Family, introducing alterations in the structural scheme. Neumann developed roofs without great reinforcement ribs acting as global stabilizing elements. His designs are characterized by surfaces that are connected

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among themselves like a continuous skin. His designs present some secondary ribs, but some authors have demonstrated that they are due to construction reasons rather than the necessity of a real structural reinforcement [3], as we can appreciate at the Residence, in Würzburg [4] or in his design for the roof in Neresheim. However, we will present in this document the Basilica of the Fourteen Holy Helpers (Vierzehnheiligen), as his most outstanding example.

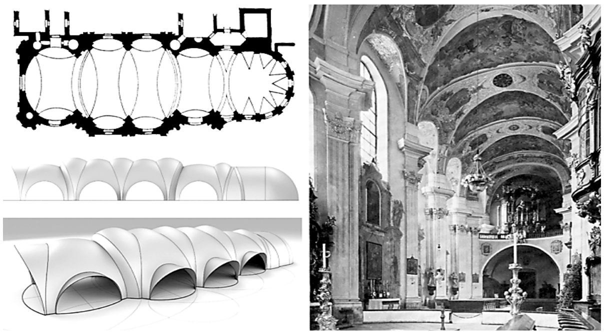

Fig. 2. St. María en Brenov. Christoph Dientzenhofer. Planta, Vista interior y modelo 3D [2]

2.1. Benedictine monastery of Banz

The church in the Benedictine monastery of Banz (Fig. 3) is organized according to Benedictine canons, that is, churches only with one nave. It counts on a sequence of three spaces that is solved through warped ribs [5]. The roof scheme consists of a sequence of longitudinal and transversal vaults that rest on powerful ribs 100 × 70 cm in section (Fig. 4). To solve the intersection, one of the vaults get the rib from its lower face and the other from its upper one, generating the typical inverted "V" solution proposed by the Dientzenhofers [6]. We can appreciate a clear commitment with the construction in its correct execution and its refined intersections.

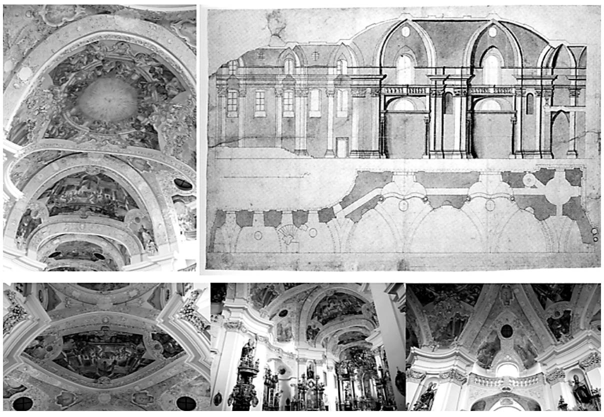

Fig. 3. Church at the Benedictine monastery of Banz. Plan by Johann Dientzenhofer. Views from the inside

It is a 55 m long, 15/20 m wide and 30 cm thick roof, made in brick masonry. The dimensions of the bricks are 14 × 28 × 5.5 cm and 1.5/2 cm lime mortar is used. There are two schemes that we can associate to the walls. On one hand, we find great-thicknesswall-pilasters at the main nave (4.3 in slenderness) and continuous walls at the final part of the nave (6 in slenderness). In this particular case, how the construction process can alternate the final deformation of the roof and thus its stress state has been studied, specially considering the effect of the load corresponding to the wooden over-roof.

The structural analysis has been made using ABAQUS 6.9-1. A mesh with solid elements has been used for the definition of the model (Fig. 5). In this case, and due to the complex form, we have choose tetrahedral elements with intermediate nodes (C3D10M). The final model has 11.5 e06 DOFs. Firstly, we have carried out an elastic linear analysis, because the building presents a good state of conservation without substantial pathologies.

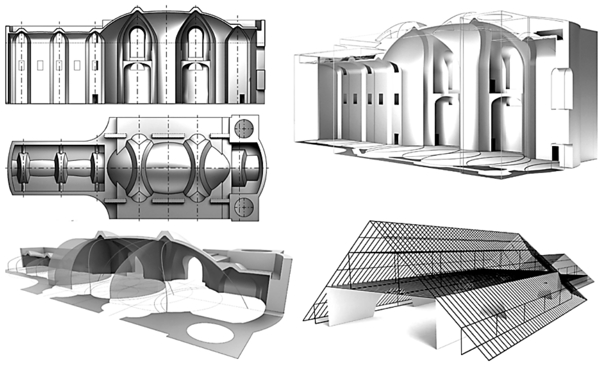

Fig. 5. Church at the Benedictine monastery of Banz. 3D model used for the structural analysis

We have considered the following mechanical properties for all the analysis that we present in this document [7]:

$$E\ (\text{MPa}) = 2000;\ \upsilon = 0.2;\ \rho\ (\text{Kg/m}^3) = 1700$$

In the case of the Benedictine monastery of Banz, the total weight of the wooden over-roof has been estimated to be 250Tn, considering the frame showed in the picture (Fig. 5).

The most significant displacements are located in the main vault. However, the stabilization ability of the warped ribs is clear for both, the vertical and the horizontal displacements, as is showed in (Fig. 6).

Given the scale of this example, the proposed solution count on an adequate structural behaviour without high tension values (Fig. 7). The use of great wall – pilasters essential

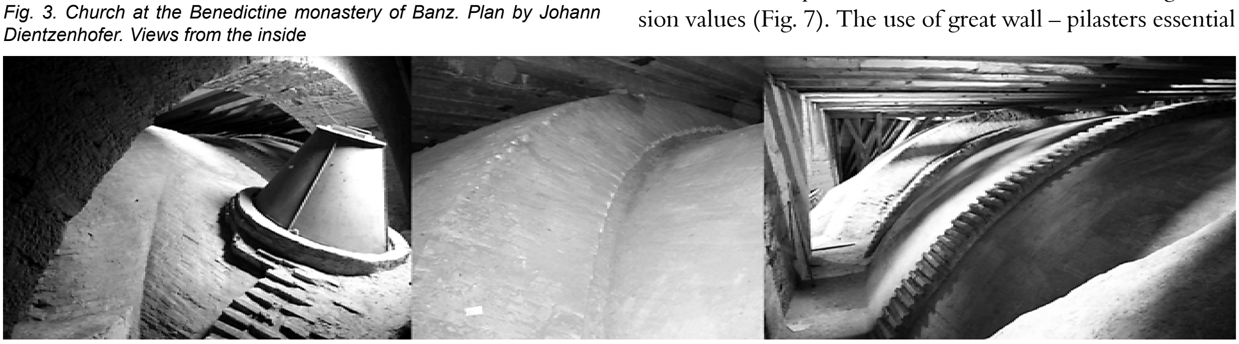

Fig. 4. Church at the Benedictine monastery of Banz.Brick masonry roof viewed from the extrados

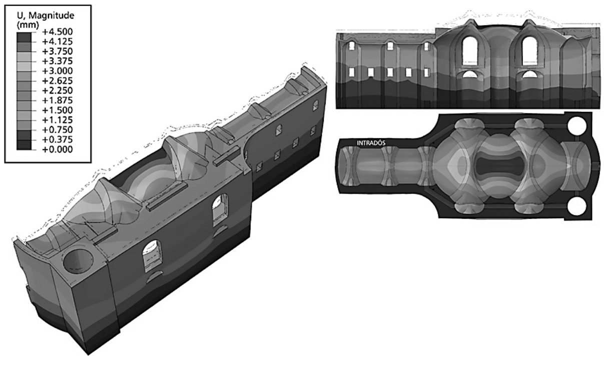

Fig. 6. Church at the Benedictine monastery of Banz. Maximum displacements

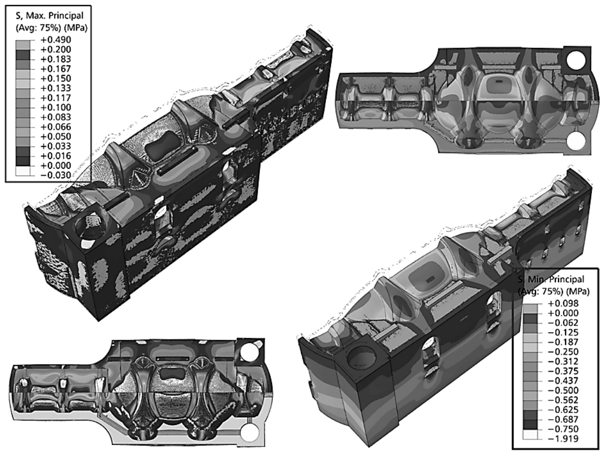

Fig. 7. Church at the Benedictine monastery of Banz. Maximum tension (SMax) and compression (SMin) stresses

for the good behaviour of the brick masonry roof, which results hard confined by the warped ribs. Due to the greater rigidity of these warped ribs, compressions flow mainly through their thickness toward the wall-pilasters.

The wooden overroof is not deciding in the stability, but contributes to the lowering of horizontal displacements (about 7%), without detriment to compressions.

The structural skeleton used works out very pertinent, because reduces tensions, confines the main vault and provides an additional and beneficial rigidity for the roof. The structural skeleton count on the elements where higher compressions are concentrated.

The tension stress states are very low, nearly null at the vaults of the main nave and a little higher at altar nave due to its lower curvature.

3. BALTHASAR NEUMANN. BASILICA OF THE FOURTEEN HOLY HELPERS (VIERZEHNHEILIGEN)

Vierzenheiligen [8] is the highest scale example (65 × 40 m in plan). It presents several features that make it especially interesting for our study: on one hand, its plan, organized from a latin crux (Fig. 8); on the other hand, its high scale, with a main vault that span more than 16m, combined to a especially complex geometry and, finally, the construction solution that was proposed.

The spatial composition consists in a series of longitudinal and transversal oval in plan vaults with a transept crown with two hemispheres (Fig. 9) [9]. The roof has no edges at the joints between longitudinal and transversal vaults. It is a continuous surface, although the intersection between two cylinders of different diameter is marked and can be seen from the inside.

The 30 cm thick roof is made using brick masonry at the base of the vaults, a sedimentary stone called Tuffstein [10] at the rest of the roof and lime mortar in both masonry (Fig. 10). The adopted solution presents no kind of structural reinforcement [3]. According to a previous analysis of the wooden overroof, its weight has been estimated at 500 Tn.

The deformation of the walls towards the inside of the basilica is due to their self-weight, although this deformation was very probably corrected during the construction process, resulting undeformed walls which are initially stressed because of their self-weight. This is the state we have considered at the moment of the application of the load corresponding to the roof.

According to this hypothesis, and to assess the importance of the horizontal load of the roof on the walls, we next present the displacements due to the complete masonry load on the undeformed walls (Fig. 11). In the case of the Basilica of the

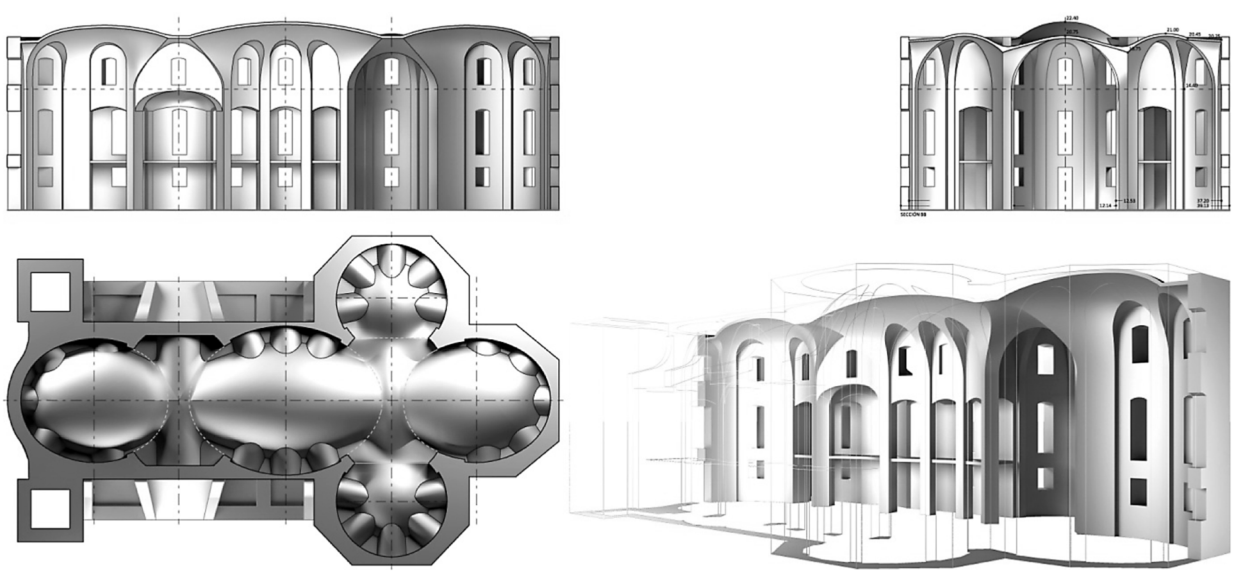

Fig. 8. Basilica of the Fourteen Holy Helpers. 3D model for the structural analysis

Fig. 9. Basilica of the Fourteen Holy Helpers. Views from the inside and detail of the warped rib as the intersection of two cylinders of different diameter [9]

Fig. 10. Basilica of the Fourteen Holy Helpers. Views from the extrados

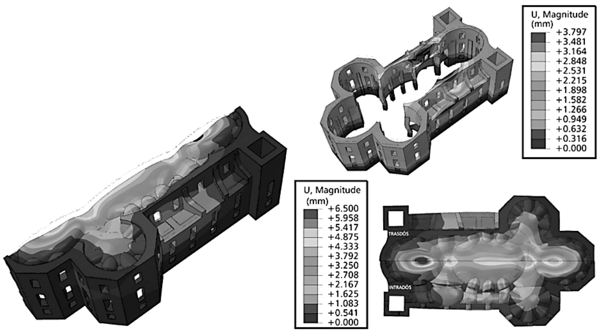

Fig. 11. Basilica of the Fourteen Holy Helpers. Maximum deformation of the walls and deformation of the roof over the undeformed walls, considering the stress state due to the self-weight

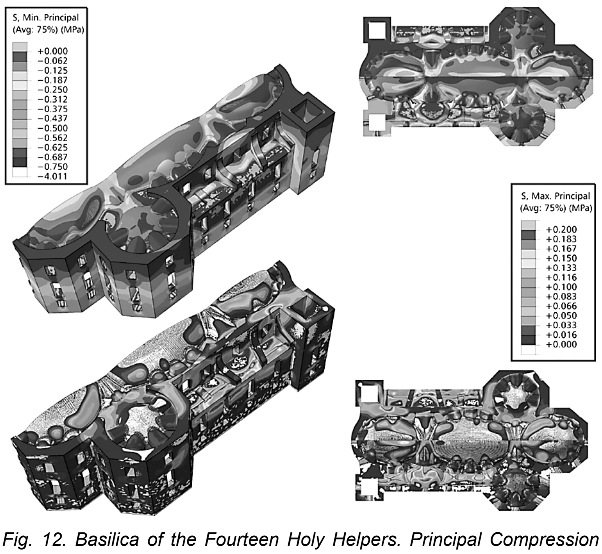

Fig. 12. Basilica of the Fourteen Holy Helpers. Principal Compression (SMin) and Tension (SMax) Stresses

Fourteen Holy Helpers, if we analyzed its complete model, we would be led to wrong conclusions, being essential the consideration of the different steps of the construction process.

As can be appreciated, we must be careful to consider deformations due to the self-weight of the walls. In this state, they present horizontal displacements which significantly modify the structural behaviour of the roof that rests on them.

We have simulated the construction process according to two steps and some substantial differences in the results obtained respect to the analysis with only one constructive step can be found. We mainly appreciate an increase of tension at the transversal vaults.

Respect to the principal compression stresses, there is also an increase respect to the one-step analysis. The stress concentration level is higher, as we can see in areas next to the symmetry plane, including the consequent decrease of this stress in the corresponding areas of the intrados. There is also an increase of compression stresses at the base of the vaults, although it is mainly located along the intersection edge between the different vaults. In any case, the obtained values are very far from the established limit one of 2 Mpa.

4. CONCLUSIONS

We can define the roofs presented in this study as light roofs, due to their small thicknesses. However, they are also shapes with features and properties that make possible defining them as elastic roofs, with a high deformation capacity and easily changeable due to external stimulations which can modify their structural behaviour in a significant manner.

We detect a certain improvement of the stress state thanks to the plastic properties that present the brick masonries used here. In this sense, stands out the thick lime joint that, thanks to its high setting times, allows some stress redistributions in the first moments after the removing of the shore, adapting in this way the shape to situations with a more homogeneous stress states and therefore avoiding local concentrations of stresses.

It is clearly evidenced the importance of making a numerical simulation having in consideration the different building stages, especially for been the masonry roof one of the last elements to be built. Therefore, it is necessary to make deformation studies to detect which of them have not got any sense in a simulation made by numeric methods.

There are other aspects that also collaborate in the good structural behaviour mentioned before. The roof organization by longitudinal vaults alternated with transversal vaults allows, in some manner, keep confined the different vaults and minimizing their deformations.

It is also clearly reflected the importance that present the tension stresses in opposition to the compression ones, not being these last ones relevant because of its magnitude.

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Abstract

Shells are forms that base their behaviour in their geometry. From Antiquity, the art of building is conceived as the ability to span and grew with little pieces, like bricks and stones, until great spaces could be closed. Arches, vaults and domes work mostly because their geometrical configuration and not so much because their intrinsic strengthening.

The complexity of Baroque domes has not been studied enough until now, at least in terms of structural analysis in conjunction with geometrical concepts. This is the reason because we have carried out a research in which geometry is considered as structure. We have studied a wide series of churches of the German Baroque and we present here two of them that maybe are the most surprising: the Chapel in the Monastery of Banz, by Johann Dientzenhofer, and The Fourteen Saints Basilica, by Balthasar Neumann.

Their complex roofs, checked in situ, have been modelled using a 3D parametric design application and analysed by Finite Elements Analysis Method using ABAQUS.

The main aim of this paper is to demonstrate that the basis of the optimal behaviour is the special geometry considered for each design. These geometries allow consider only membrane stresses instead of shell stresses. Bending is really acting in border and edges and not in the general surface as occurs in concrete and steel shells. Baroque was the inventor of membranes as we actually conceive them, with examples that present such complexity that none of the great concrete builders have tried to build.

Streszczenie

Sklepienia łupinowe są formami, których zachowanie oparte jest na ich geometrii. Już od starożytności, sztuka budowania polegała na umiejętności tworzenia elementów konstrukcyjnych z niewielkich fragmentów, takich jak cegły czy kamienie, co pozwalało na zamknięcie wielkich przestrzeni. Łuki, sklepienia i kopuły funkcjonują głównie z racji swojej konfiguracji geometrycznej, a nie z powodu wewnętrznego wzmocnienia.

Złożoność barokowych kopuł nie została jak do tej pory odpowiednio przestudiowana, przynajmniej pod względem analizy konstrukcji w połączeniu z koncepcjami geometrycznymi. Z tego też powodu przeprowadziliśmy badania, w których geometria została potraktowana jako konstrukcja. Przestudiowaliśmy szereg kościołów z okresu niemieckiego baroku i prezentujemy tutaj dwa z nich, które wydają się być najbardziej interesujące: Kaplicę z klasztoru w Banz, zaprojektowaną przez Johanna Dientzenhofera, oraz Bazylikę Czternastu Świętych Wspomożycieli Balthasara Neumanna.

Ich złożone przekrycia, zbadane in situ, zostały zamodelowane z wykorzystaniem projektowania parametrycznego w 3D i przeanalizowane za pomocą metody elementów skończonych z zastosowaniem programu ABAQUS.

Celem niniejszej pracy jest zademonstrowanie, iż podstawą optymalnej pracy konstrukcji jest geometria jej poszczególnych elementów, specyficzna dla każdego projektu. Ta geometria pozwala brać pod uwagę jedynie naprężenia membranowe zamiast naprężeń powłokowych. Zginanie oddziałuje na granicach i brzegach, a nie na całej powierzchni, jak to ma miejsce w przypadku powłok betonowych i stalowych. W baroku wynaleziono membrany w naszym rozumieniu tego pojęcia, a ich przykłady mają tak złożony charakter, iż żaden z twórców wielkich budowli z betonu nie próbował nic podobnego stworzyć.

Alberto Ayensa Pardo[1], Beatriz Beltrán Pellicer[2],
Elena Ibarz Montaner[3], Luis Gracia Villa[4]

A methodology for the assessment of historical structures based on finite element models

Metodologia oceny historycznych konstrukcji w oparciu o modelowanie metodą elementów skończonych

Keywords: Historical structures, Finite element, Masonry, Stone, Brick, Arch, Vault, Flying buttress

Słowa kluczowe: historyczne konstrukcje, Finite Element, kamieniarka, kamień, cegła, łuk, sklepienie, przypora

1. INTRODUCTION

The most common methodology applied currently to verify the safety of masonry historical structures is the graphic statics under the limit analysis [1]. It is based on a comparative analysis between the equilibrium situation of the structure and its limit situation considering the stability. The result of this analysis is the geometrical factor of safety. It is an indicator of the safety of the structure considering its stability when the loads are applied and fixed supports are considered (without foundation settlements, spread of the abutments, etc.).

The equilibrium situation of the structure, applying the current methodology, is determined considering the equilibrium equations, but not the compatibility and the material constitutive equations. Therefore, the calculated result only depends on the geometry of the structure, the applied loads and the consider reactions at the supports. The solution of the equilibrium equations system is obtained applying the graphic statics [2], determining the funicular polygon that equilibrate the loads acting on the structure and, from it, it is deduced the thrust line. It is assumed the following hypothesis [3]:

- The masonry blocks are rigid elements and it is assumed that the material failure in compression is not possible.
- The masonry tensile strength is null. It is not considered the cohesion due to the mortar located between blocks.
- The sliding between blocks is not possible.

The thrust line, obtained from the structural analysis, allows knowing the minimum geometry of the structure necessary to be in situation of equilibrium and, comparing the minimum geometry with the real geometry it is determined the geometrical factor of safety. It is usually considered that the masonry historical structures are safe when their geometrical factor of safety is equal or higher than 2, but this value varies according to the construction style. In case of masonry bridges, both the graphic statics under the criterion of the limit analysis and finite element models that simulates the bridges behavior [4] are currently applied. In this last case, the stone blocks, fills and pavement of the bridges are modeled and over them it is applied the loads to calculate the bridge response to verify their safety considering the stability.

In this work a methodology based on the finite element method to analyze historical structures is presented, considering the following hypotheses:

- There is no tensile strength at the joints between blocks.
- Shear strength of the joints is limited by the shear strength of the mortar and the static frictional coefficient in the interface block-mortar-block.
- Brittle facture occurs in tension as the elastic limit in tension is reached.
- Plastic deformation and crushing appear as the elastic limit in compression is reached.

Applying this methodology, the following aspects can be verified:

- Situation of equilibrium of the structure.
- Stresses in the different elements of the masonry.
- Possible sliding between blocks.
- Capacity of the structure to resist foundation settlements.

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– Influence of the degradation of some blocks considering the durability, the decreasing of their density and their elastic modulus.
– Consequence of the installation of reinforcements (actives or passives) in the structure and the stresses in them.
– Effect of the cracks seal with non shrinkage mortar.
– Soil bearing capacity under the foundations.
– Conditions in which the structure collapses and the collapse progress.

2. GENERAL EXPOSITION

The proposed methodology to analyze and evaluate masonry historical structures of monumental buildings can be summarized in the following steps:

Study of the building "on site" to obtain the necessary information: all the dimensions and levels of the structure, its location coordinates, high from the level sea and wind exposure, the material properties and the soil bearing capacity.

Study of the loads acting on the structure. Considering the current standards, the actions [5] and their combinations [6], which depend on the limit state to verify, can be obtained. The simultaneity factors to apply are defined in the current standards and the safety partial factors to consider are shown in table 1.

Table 1. Safety partial factors

Persistent or transitory situation			Accidental situation		
Stability of the structure	$\gamma_{G,stb}$ = 0,90	$\gamma_{Q,dst}$ = 1,80	Stability of the structure	$\gamma_{G,stb}$ = 0,90	$\gamma_{Q,dst}$ = 1,20
Sliding between blocks	$\gamma_{G,stb}$ = 0,90	$\gamma_{Q,dst}$ = 1,50	Sliding between blocks	$\gamma_{G,stb}$ = 0,90	$\gamma_{Q,dst}$ = 1,20
Masonry resistance	γ_G = 1,35	γ_Q = 1,50	Masonry resistance	γ_G = 1,00	γ_Q = 1,00
Reinforcement resistance	γ_G = 1,35	γ_Q = 1,50	Reinforcement resistance	γ_G = 1,00	γ_Q = 1,00
Soil bearing capacity	γ_G = 1,00	γ_Q = 1,00	Soil bearing capacity	γ_G = 1,00	γ_Q = 1,00

Creation of the geometric model. The scaled masonry structure is graphically represented using a CAD application. The geometrical model consists in an assembly of pieces in contact and each piece can represent a single block of the structure or a group of blocks.

Preparation of the model to analyze the structure. All the pieces of the geometric are meshed at this step. It is also assigned to the assembled pieces the boundary conditions (loads and imposed displacements), the contact behaviour between pieces and the material properties (density, Young's modulus, Poisson's ratio, etc.).

Calculation of the model applying the finite element method. The method formulation [7] allows to avoid the imposed restrictions in the graphic statics.

Evaluation of the results. According to figure 1, after the analysis, if there is an equilibrium solution the safety of the structure can be determined verifying several parameters. On the other hand, if do not exist such solution, it means that the structure will collapse under the imposed boundary conditions on the model and then, the collapse evolution can be determined applying a progressive analysis.

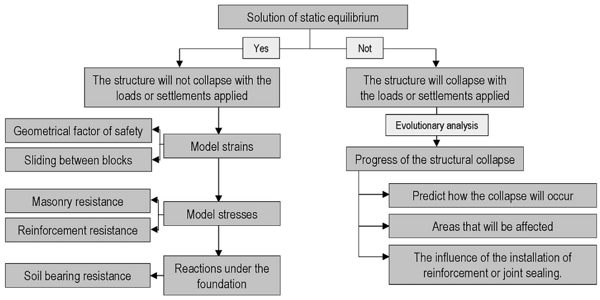

Fig. 1. Verifications that can be checked applying the proposed methodology

3. METHODOLOGY DEVELOPMENT

3.1. Creation of the geometric model

The pieces of the geometric model should be created using a CAD application and considering two possible criteria. Each piece can represent a single block of the masonry structure or, on the other hand, a group of blocks. The correct criteria to create pieces that represent a group of blocks can be summarized in the following points:

– The blocks with common joints where all the contact surfaces are in compression can be represented in the geometric model by just one piece. The objective is to find groups of blocks with rectangular or trapezoidal contact pressure distributions between them, considering their behaviour as only one elastic solid. This situation appears when the thrust line lies inside the kernel of the blocks section. It the above condition is not satisfied, the joint between blocks will be cracked, so these blocks are associated to different pieces in the geometric model.
– The masonry blocks located at geometric discontinuities must be associated to different pieces of the geometric model, e.g. where an abutment changes its direction or its thickness, an abutment or a column supports an arch, a vault or a flying buttress, etc.
– It is recommended, in the safety side, to group the blocks of domes, semi-domes, etc. in pieces with segments shape according to the "cuts method" [1] of the graphic statics. The vertical cracks generated in the geometric model are necessary to consider the typical vertical cracks shown in the figure 2 appearing in domes due to the horizontal tensile stress in the base of this type of structures.

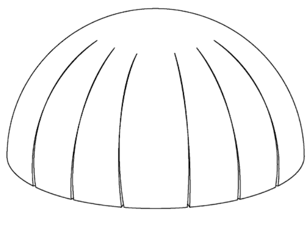

Fig. 2. Typical vertical cracks in the base of the domes

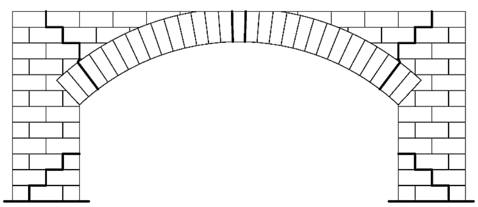

Fig. 3 Typical cracked areas in abutments that support arches

– It is also recommended to separate in different pieces the blocks located in areas where high shear loads and low axial loads of compression are expected. It is necessary to verify the possibility of sliding between the pieces of the geometrical model and, therefore, between the blocks of the masonry structure.
– Finally, it is important to create a joint at the base of the abutments of the geometric model as shown in the figure 3 (forming 45° with the horizontal plane) to take into account the possible cracking that usually appears in the bases of the abutments. When the thrust line does not go through the kernel of the section of the abutment base, a crack appears initially in the intrados of the base and later, due to the high self weigh of the abutment, the abutment is broken and a new crack appears forming 45° with the base [8].

The above criteria are recommended to simplify the geometrical model in order to achieve reliable results with the minimum computational cost. However, it is not compulsory to simplify the model but, in that case, the computational cost will increase a lot and the results will be the same.

3.2. Preparation of the model

Once the geometric model has been created, it is necessary to assign the boundary conditions to the pieces, that is to say, the loads that act on the structure, the supports below the foundations, the imposed displacements that simulate the effect of settlements, etc. It is also necessary to mesh and to assign the material properties to each piece and finally, to define the contact behaviour between pieces.

The loads and their combinations are indicated in the current standards applying the safety partial factors shown in the table 1 and, on the other hand, the settlements that should be considered in the foundation to verify the safety of the masonry structure are:
– A maximum absolute settlement: δ_{vert} = 25 mm.
– A maximum relative settlement: δ_{vert} = L/2000, where L is the distance between the considered points to obtain the máximum relative settlement between them.
– The real settlements observed in the structure during the study of the building "on site".

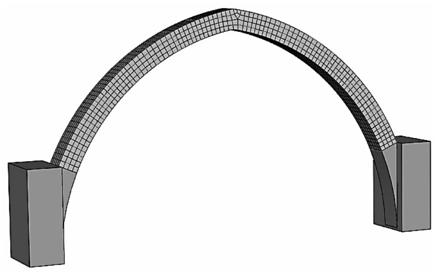

Fig. 4. Equilateral arch meshed

The recommended elements to mesh the pieces, as shown in the figure 4, are quadratic bricks of 20 nodes or quadratic tetrahedra of 10 nodes where the geometry is complicated.

The material properties that should be assigned to the meshed pieces should be obtained considering parameters determined from structural tests "on site" or specialized bibliography. These necessary material properties are:
– Masonry density (ρ).
– Young's modulus of the masonry (E_m), calculated as:

$$E_m = E_b \cdot \frac{1+\alpha}{1+\alpha \cdot \beta \cdot (1+\varphi)} \tag{1}$$

where the E_b is the Young modulus of the masonry blocks, α is the ratio of the average thickness of the mortar bed joints and the average high of the blocks, β is the ratio of the Young's modulus of the blocks and the Young's modulus of the mortar and φ is a parameter that takes account the masonry creep which can be considered null analizing historical structures. The values of the α and β parameters can be obtained applying the following ecuations:

$$\alpha = \frac{h_m}{h_b} \qquad \beta = \frac{E_b}{E_m} \tag{2}$$

– Poisson's coefficient of the masonry (ν).
– Design value of the masonry compressive strength ($f_{m,d}$). It is the limit from which the non linear behaviour of the material starts and can be obtained applying the Ohler formula [9]:

$$f_{m,k} = \frac{a \cdot f_{b,k}}{1+10 \cdot b \cdot \alpha} \qquad f_{m,d} = \frac{f_{n,k}}{\gamma_R} \tag{3}$$

where γ_R = 4 is the partial factor for the material properties, $f_{b,k}$ is the characteristic value of the blocks compressive strength and the 'a' and 'b' non-dimensional parameters can be obtained from the table 2 in function of α.

Table 2. Parameters of the Ohler formula

	a	b
∞ ≤ 0,02	1.00	2.22
0,02 < ∞ < 0,05	0.81	0.96
∞ ≥ 0,15	0.66	0.66

– Design axial tensile strength of the blocks ($f_{b,td}$). It is the limit from which the cracking of the material starts and can be calculated as:

$$f_{b,tk} = 0.05 \cdot f_{b,k} \qquad f_{p,td} = \frac{f_{p,tk}}{\gamma_R} \tag{4}$$

where γ_R = 4 is the partial safety factor for the material properties, $f_{b,k}$ is the characteristic value of the blocks compressive strength and $f_{b,tk}$ is the characteristic axial tensile strength of the blocks.

The contact properties that must be assigned to the contact surfaces should be as follows:
– Normal behaviour: it is disabled the possibility of interference between the pieces, that is to say, each piece can transfers thrusts to next pieces and cannot penetrate in them. It is also allowed the separation of the contact surfaces after the contact.
– Tangential behaviour: it is allowed the sliding between pieces if the maximum friction load is exceeded at their contact surfaces considering an static frictional coefficient.

After the preparation of the model, it is calculated applying the finite element method. It is recommended to start the analysis procedure fixing all the pieces of the model to ensure an initial mathematical convergence to an equilibrium solution (if it exists). After that, assigning contact conditions instead of the fixing conditions in successive iterations, the solution of the model considering the contact conditions recommended in the proposed methodology will be obtained.

3.3. Results analysis

The safety of the structure considering the stability is verified obtaining equilibrium solutions in all analyzed cases. The scope of these analyzed cases should be all the possible situations of the structure during its working life (e.g. extreme loads, foundation settlements, blocks deterioration, etc.).

On the other hand, historical structures have high execution tolerances and are also characterized by their self-weight as the dominant action; therefore their design geometry is very important because their safety considering the stability assuming the execution tolerances depends mainly on the geometrical design. To take into account the effect of the execution tolerances on the safety of the structure from the stability point of view, one of the following verification criteria must be used:

- The execution tolerances can be considered in the creation of the geometrical model.
- The geometrical model can be created without the execution tolerances. Therefore they should be considered calculating the geometrical factor of safety of the structure analyzing the deformed model calculated applying the self-weight of the masonry structure on the pieces and without consider settlements on the foundation, that is to say, in the situation when the centering is removed.

The geometrical factor of safety is usually the result obtained applying the graphic statics under the limit analysis criteria. Applying the proposed methodology the geometric factor of safety can be determined from the analysis of the cracks opened between the pieces in the deformed model. Considering the ratio of the compressed area of the contact surface and the whole area of the contact surface can be determined discretely the local geometrical factor of safety at any joint (e.g. at the joint "i" the $GSF_i = 3$ when the ratio = 1, $GSF_i = 2$ when the ratio = 0.75 and $GSF_i = 1$ when the ratio ≤ 0.1). This discrete procedure to obtain the local geometrical factor of safety at any joint (GSF_i) must be changed for a new criterion that defines this factor in a continuum way considering plane flexo-compression according to the figure 5.

The geometrical factor of safety of any structural element (GSF_{elem}) can be obtained applying the equation 9 considering the local geometrical factor of safety of its critical joints (GSF_i), that is to say, of the contact surfaces of its pieces where "hinging" cracks [1] are opened. Finally, the geometrical factor of safety of the whole structure is equal to the lower value of the GSF_{elem} of the all elements of the analyzed structure:

$$GSF_i = \frac{1}{2}\frac{h_i}{e_i} \qquad GSF_{elem} = \frac{\sum_{i=1}^{n} GSF_i}{n} \tag{5}$$

where n is the number of critical joints in the structural element. In the particular case where n is null because there is not critical joints, $GSF_{elem} \geq 3$ in this element.

The safety of the structure considering the masonry resistance is verified comparing the maximum value of the compressive stress field with the design value of the masonry compressive strength ($f_{m,d}$). The safety of the structure considering the reinforcement resistance is verified comparing the maximum value of tensile stress in the reinforcement with the design value of the reinforcement tensile strength specified by the manufacturer or the current standard. Finally, the soil bearing capacity is checked considering the reactions at the foundation under the current standard criteria, e.g. the CTE-SE-C [10] in Spain.

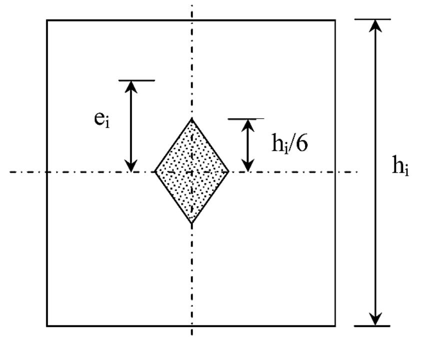

Fig. 5. Kern of a rectangular section

4. HISTORICAL MASONRY STRUCTURE ANALYSIS

4.1. Definition of the structure, creation and preparation of the model

In the figure 6 is shown the masonry structure to be analyzed applying the proposed methodology in this work. It is composed by an arch, two flying buttress and two abutments. All of them are 1 m depth. In this case, only the self-weight of the masonry acts on the structure. It has been considered a 2500 kg/m^3 of masonry density. Figure 6 shows the pieces in which the structure has been divided to create the geometrical model with a CAD application.

The following tasks were performed to prepare the model:

- Meshing the pieces with quadratic brick elements of 20 nodes and 100 mm of maximum width and assembling them how is shown in the figure 7. Abaqus program [11] was used.
- Assigning the masonry properties: Young's modulus E_m = 15000 MPa, Poisson's coefficient $\nu = 0.2$, design value of the masonry compressive strength $f_{m,d}$ = 5 MPa and the design axial tensile strength of the blocks $f_{b,td}$ = 0.25 MPa.
- Assigning also the recommended contact properties, considering a static frictional coefficient $\mu = 0.3$, to the contact surfaces.
- Applying the boundary conditions: the loads (only the self-weigh) and the supports (that depends on the analyzed case).

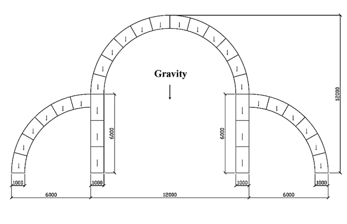

Fig. 6. Proposed structure (dimensions in mm)

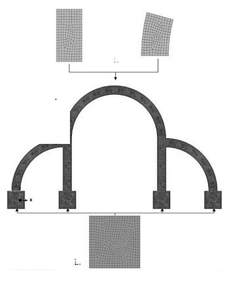

Fig. 7. Assembly of meshed pieces

4.2. Results

The deformed model in equilibrium situation applying the self-weight of the structure and without consider foundation settlements, is shown in the figure 8. It can be obtained that the GSF = 1.2 from these results, so if the proposed structure was built, it could collapse when the centering would be removed unless the structure was built perfect (without significant execution tolerances). The GSF should be ≥ 2 to ensure the stability of any historical structure taking account the instability effects due to the real imperfections. On the other hand, no sliding occurs between pieces in the analyzed situation.

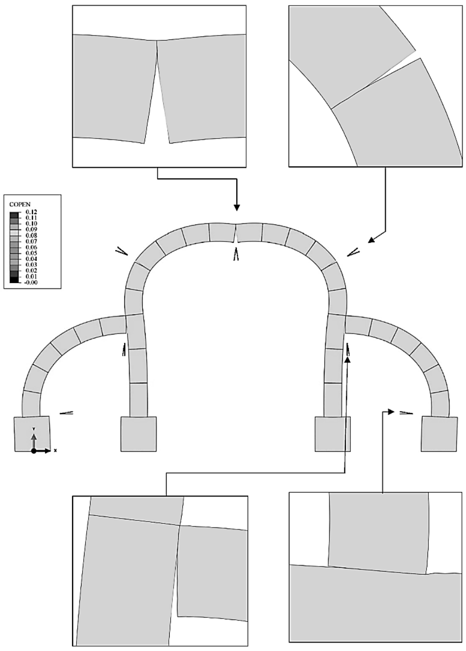

Fig. 8. Deformed model (x 2000)

The stress field of the model obtained applying the design loads and without consider foundation settlements is shown in the figure 9. The maximum compressive stress is 0.77 MPa in this case and the design value of the masonry compressive strength is 5 MPa, therefore the safety of the structure is high considering the material strength (the safety factor is 6.49).

The stress field on the deformed model obtained applying the design loads and considering a 25 mm of settlement in the left foundation, is shown in the figure 10. The maximum compressive stress is 3.14 MPa in this case, so the structural safety is reduced considering the material strength when a settlement appears (the safety factor is 1.59 in this case). Regarding the deformed model, there is a small sliding between pieces under the left flying buttress but however, its effect is not enough to collapse the structure because an equilibrium solution exists.

The model has been also calculated applying the self-weight of the structure and an arch maintenance load (without consider foundation settlements) to verify the structural safety if the arch maintenance is carried out.

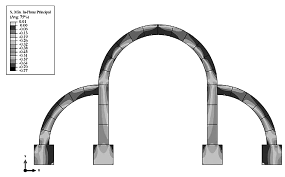

Fig. 9. Stress field in the model without Settlements

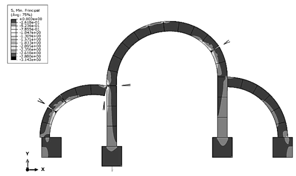

Fig. 10. Stress on the deformed model with 25 mm settlement

The maintenance load is 15 kN acting on the keystone (the keystone is composed by the two upper pieces of the model) and there is not equilibrium solution in this case, so the structure collapses. The progress of the structural collapse is shown in the figure 11, obtained from an evolutionary analysis.

Finally, the model is reinforced with a temporally steel bar to avoid the structural collapse when the arch maintenance is carried out. The position of the steel bar has been determined analyzing the figure 11 (c) which shows the collapse mechanism that should be avoided. The stress field on the deformed reinforced model obtained applying the design loads and without considering foundation settlements, are shown in the figure 12. The maximum compressive stress in the masonry is 1.29 MPa in this case, so the structure safety is high considering the material strength (the safety factor is 3.87) when the arch maintenance is carried out. Regarding the deformed model, no sliding occurs between pieces in the analyzed situation. The scope of this work does not include the verification neither the soil bearing capacity nor the tensile yield strength of reinforcement.

5. CONCLUSIONS

In spite of the existing analogies between the developed methodology in this work and the graphic statics under the

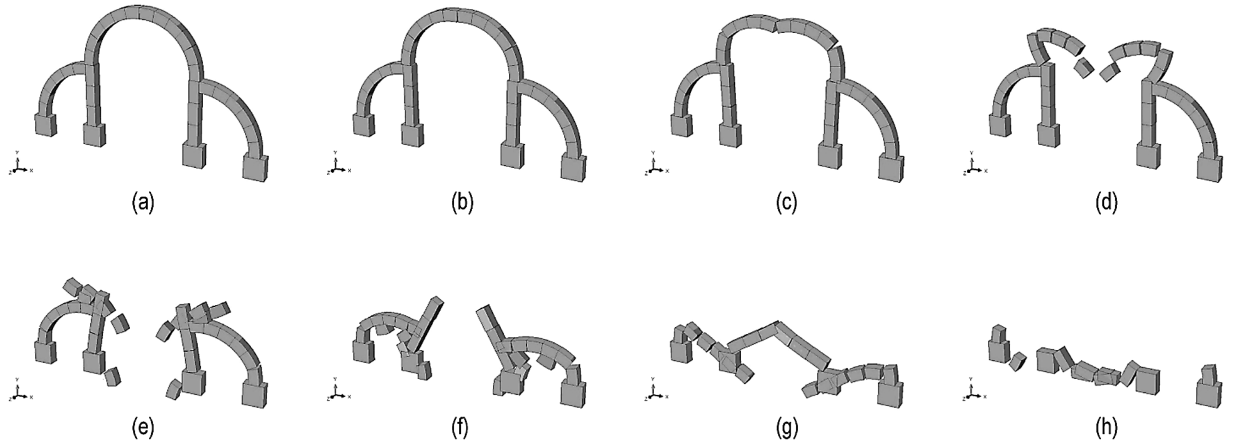

Fig. 11 Progress of the structural collapse

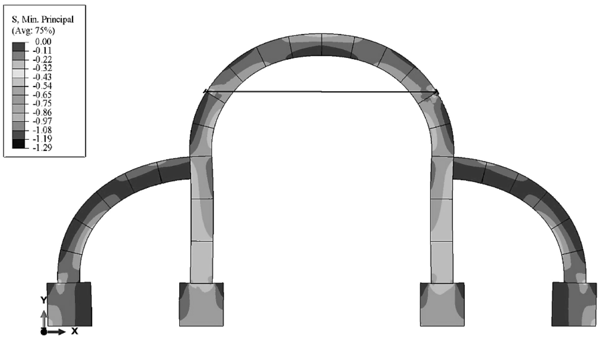

Fig. 12 Stress field on the deformed reinforced model (x 2000) due to the maintenance load

limit analysis criteria, this last methodology has several limitations when comparing with the proposed methodology. The main advantages are:

Addresses much more situations: It can consider foundation settlements, blocks deterioration, different types of material behaviour, active or passive reinforcement, joint sealing with non shrinkage grout, etc.

Besides the geometrical factor of safety, the proposed methodology allows to verify the structure safety considering the failure by sliding between blocks, the material failure and the loss of the overall stability due to foundation settlements, etc.

The collapse progress can be predicted in case that the structure loss the stability.

Tree-dimensional masonry structures can be analyzed (the graphic statics method can analyze only planar structures).

Due to the reasons exposed above, the methodology developed in this work represents a way for the assessment of historical monumental structures with high level of reliability in order to to determine their structural safety in different situations.

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[11] Abaqus computer program. http://www.3ds.com/

Abstract

This work proposes a methodology to analyze masonry historical structures (stone or brick). They are mainly characterized by their self-weight which is the dominant action that acts on them, such us churches, cathedrals, etc. The proposed methodology is based on the analysis of finite element models that simulate the real structure behaviour. Therefore, much more information can be obtained about the structure safety than applying the usual methodology (the graphical statics considering the limit analysis). Using the proposed methodology, the structure response can be determined applying some loads on the model and considering some support conditions that simulate foundation settlements. The effect of structural reinforcement, wear of some blocks, non linear properties of the materials, etc. can be also considered. The safety level of the studied structure is evaluated with the obtained analytical results.

Streszczenie

W niniejszej pracy zaproponowano wykorzystanie metody elementów skończonych do analizy historycznych konstrukcji murowanych (kamienia lub cegły). Zasadniczą cechą tych konstrukcji jest ich ciężar własny, który stanowi w ich przypadku podstawowe obciążenie np.: kościoły, katedry itp. Zaproponowana metodologia jest oparta na analizie metodą elementów skończonych modeli, które symulują zachowanie rzeczywistej konstrukcji. Pozwala to na uzyskanie większej ilości informacji dotyczących bezpieczeństwa konstrukcji, niż przy użyciu zwykłych metod (statyka wykreślna z uwzględnieniem analizy stanów granicznych). Przy wykorzystaniu proponowanej metody, reakcja konstrukcji może być określona poprzez zwiększenie obciążenia na modelu i przebadanie warunków podparcia, które symulują osiadanie fundamentów. Efekt wzmocnienia konstrukcji, zużycie materiałów, ich nieliniowe właściwości, itp. mogą również zostać uwzględnione. Poziom bezpieczeństwa badanej konstrukcji jest oceniany na podstawie uzyskanych wyników analiz.

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Testing timber beams after 130 years of utilization

Badania belek drewnianych po 130 latach użytkowania

Keywords: Reuse, Laboratory tests, Timber beams

Słowa kluczowe: powtórne wykorzystanie, badania laboratoryjne, drewniane belki

1. INTRODUCTION

Use of wood in buildings construction in Poland, after years of underestimation of this material, has been used more and more often. In the past obtaining wood was not difficult due to quite large afforestation of Poland. That is why the problem of reuse of whole timber beams are practically unknown in our country. In recent years the prices of this material in Poland as well as in the whole Europe have been rising. At the same time more and more structures with timber construction are being demolished or thoroughly reconstructed and renovated. Old timber beams are usually thrown out at the waste dumps or used as heating fuel. Very often these beams are in good technical condition and they could be used in other buildings however, the tests confirming their good technical properties are missing. That is why testing of such solutions has been undertaken. For this purpose timber floor beams were obtained for laboratory tests after 130 years of utilization in the floor over the ground floor (Fig. 2) from monumental building under renovation in Połomia (Fig. 1).

The subject building was constructed in 1878. It functioned as a presbytery until the 50s of 20th century. Then it was utilised by the commune as: a commune office, a kindergarten and in the 80s as a primary school. In 1995, after the new primary school was built in Połomia, the building was abandoned. The building was not utilised for 15 years and underwent destruction process. Its demolition was even considered. However, in 2010 on February 19th the building was entered to the national register of historic monuments under number A/295/10. At the moment it is being under renovation. The roof and floors have been replaced and the rooms will be used for new purposes. The subject building has got basement under part of it and it consists of three floors and usable attic.

Fig. 1. The photo of „old" school in Połomia

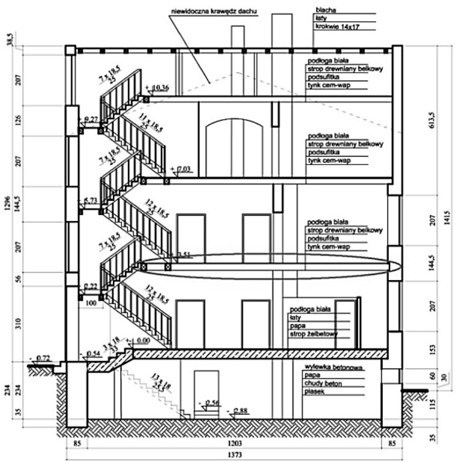

Fig. 2. Cross-section of the school in Połomia

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2. CHARACTERISTICS OF TESTED BEAMS

Eight 130-year-old beams of the length between 3.03 and 5.11 m were obtained for laboratory testing. The beams first played the role of floor bearing elements over the ground floor. Original floor was first made as beam-framed floor with sound boarding and ceiling. Testing elements delivered to the laboratory had got vivid restages of old floor – in the form of nails, sound boarding lathes and other impurities.

During the in-situ inspection the following wood properties were determined:

– The looks of the wood – beams (130-year-old) became greyish, clearly grey beams surfaces. There are no distinct protection coats (impregnants, paint coats) on the beams. Surfaces of all beams are rough and matt.
– The smell of wood – smell characteristic for wood cannot be sensed, just unpleasant smell of dust.

After the beams surfaces were cleaned it was noticed that the beams were made of spruce timber. Technical condition of the obtained beams was in the majority of cases good with visible longitudinal cracks. In some of the beams insects feeding marks were present as well as surface biological corrosion visible in the spots where beams rested against the walls. Wood tested humidity ranged 12-15%.

Due to the beams length, five wooden beams of the length ranging from 4.48 m to 5.11 m were used for bending testing. The remaining three sections of beams less than 4 m long were used for compression testing. The testing results, though, are not presented in this paper. Table 1 presents dimensions of the beams as well as brief description of technical condition of the beams used for testing. Three of the beams were subject to the test in the scheme of the beam loaded with two forces with the supports spacing $l = 4.5$ m, whereas two other in analogical scheme with supports spacing $l = 4.0$ m. Preparation to testing of the obtained 130-year-old beams consisted in removal of laths supporting sound boarding, removal of old nails, cleaning and inventory.

Table 1. Description of beams used for bending testing

No.	Designation	Dimensions [m]			Comments
		width	height	length	
1	BD2	0.195	0.250	4.93	Generally in good technical condition, visible longitudinal cracks as well as insects feeding marks.
2	BD5	0.208	0.256	5.02	Generally in good technical condition, longitudinal cracks on one side surface.
3	BD6	0.212	0.245	5.11	Generally in good technical condition, visible insignificant longitudinal cracks.
4	BD3	0.202	0.240	4.53	Generally in good technical condition, visible longitudinal sloping cracks. On one head insignificant biological corrosion.
5	BD8	0.206	0.262	4.48	Generally in good technical condition, visible insignificant longitudinal cracks.

3. TESTING OF OLD BEAMS CONDITIONS AND PROPERTIES

3.1. Testing of floor beams in natural scale

3.1.1. Testing stand and description of tests

As it was mentioned above, bending tests were carried out for two different supports spacing. Beams BD2, BD5 and BD6 were tested with supports sparing equal $l = 4.5$ m whereas beams BD3 and BD8 were tested with supports sparing equal $l = 4.0$ m. Actual length of tested beams decided about supports spacing. Scheme of the testing as well as the view of testing stand have been presented correspondingly in Figs. 3 and 4. During the testing force, deflections and strains on upper and bottom surfaces of the beam were measured.

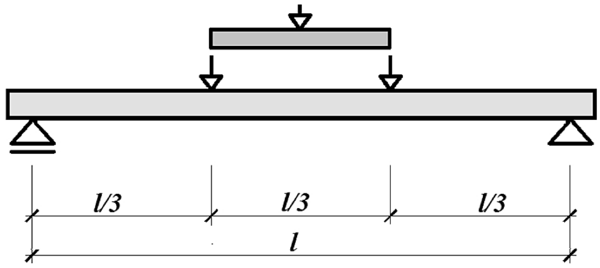

Fig. 3. General testing scheme

Fig. 4. View of the bending testing stand

3.1.2. Results of testing beams in natural scale

Testing results have been presented in Table 2. It includes values of the destructive force for particular testing models as well as value of the deflection at the destruction moment. For easier comparison, because of different support spacing, Table 2 includes also value of bending moment as well as stresses values at the destruction moment.

Table 2. Beams load bearing capacity

Beam designation	Mean dimension of beam cross-section [m]		Support span [m]	Destructive force [kN]	Moment at destruction [kNm]	Bending stresses at destruction [MPa]	Deflection [mm]
	Width	Height					
BD2	0.195	0.250	4.5	73.94	55.455	27.301	49.40
BD5	0.208	0.256	4.5	89.62	67.215	29.585	44.43
BD6	0.212	0.245	4.5	78.495	58.871	27.758	48.36
BD3	0.202	0.240	4.0	99.53	66.353	34.217	46.51
BD8	0.206	0.262	4.0	130.17	86.780	36.821	48.98

To estimate the class on the wood standard PN-EN 384:2004 [1] has been used. According to this standard determination of the mechanical properties of the wood has been carried out of full-size elements.

Strength characteristic value has been calculated with the use of the formula:

$$f_k = f_{05} \cdot k_s \cdot k_v \quad (1)$$

where:
f_{05} – the value of 5% of quantile,
k_s – correction factor regarding number and size of sample,
k_v – correction factor regarding machine sorting.

Value f_{05} is determined based on the formula:

$$f_{05} = f_r \quad (2)$$

where:
f_r – value corresponding to 5% of quantile of the ranged testing results

Due to the small number of samples 5% quantile has been determined based on the formula:

$$f_{05} = f_{śr} - t_\alpha \cdot s \quad (3)$$

where:
t_α = 1.64 (statistical ratio for normal distribution corresponding to probability p = 0.95%),
s – standard deviation for current sample = 4.193 MPa
where: H – sample height in mm.

Table 3. Strength properties of the tested timber

	Mean strength $f_{śr}$ [MPa]	Standard deviation s [MPa]	Correction factor k_s [–]	Correction factor k_v [–]	Characteristic strength f_k [–]
Bending	31.136	1.61	–	1.0	24.26
Bending	31.136	1.61	0.77	1.0	18.67

Based on the obtained tests result, due to the small number of the beams, only current class of wood has been estimated.

Without taking factor k_s (correction factor regarding number and size of samples) into account the wood of the beams can be ranked as class C24 according to PN-EN 338 [2]. As is has been presented in Table 3 $f_k = f_{05}$ = 24.26 MPa. However, it needs to be noted that all tested beams underwent destruction at bending stresses greater than 24 MPa.

If factor k_s, which for small sample equals 0.77, is considered, then the wood shall be classified as C18.

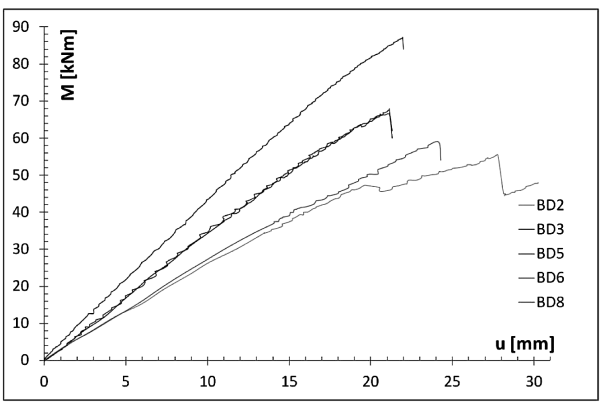

Fig. 5. Dependence: moment – deflection

Dependence of the deflection „u" on the applied load has been presented in Fig. 5 as a relationship: moment – deflection. This way of presentation results from the use of various spans in testing old timber beams and enables comparison of particular beams. As it has been shown in diagram presenting relationship moment – deflection for tested 130-year-old beams is clearly linear, different than in testing of new timber. Based on the extensometer measurements results, neutral axis lowering has not been observed either. Destruction of the beam usually happened in a violent way but it was preceded with classical cracks. Exemplary forms of destruction have been presented in Fig. 6.

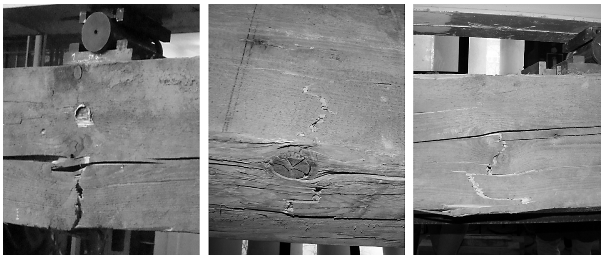

Fig. 6. Forms of beam destruction – from the left beams: BD6, BD5, BD2

3.2. Material testing on little samples

3.2.1. Testing description

In strength tests the results depend on the samples dimensions. Little samples without any defects give greater strength values than in case of bigger samples, in which wood defects are unavoidable. According to Polish Standard PN-EN 384:2004, testing of the wood strength properties shall be carried out with the use of full-dimensional elements. In case of wood properties determination for the existing constructions it is practically impossible [3]. That is why strength properties testing has also been carried out on small samples without defects in accordance with "the old" standard PN-77/D-04103, which is still very often used particularly for determination of the strength parameters of the existing structures. According to these standards samples for bending testing should have dimensions 20 × 20 × 300 mm and for compression testing 20 × 20 × 30 mm. The tests for bending were carried out on 40 small beams taken from earlier tested floor beams.

3.2.2. Results of bending and compression testing on small beams without defects

Due to quite large number of samples, only averaging test results for small samples with standard deflection have been provided and then they were recalculated for full-dimensional elements with the use of formulas (1), (2), (3) provided under item 3.1.2 of this paper (as for full-dimensional elements) and then size of samples was taken into account (formula (4)) as well as dissimilarity of testing scheme (formula (5)). Testing results have been presented in Table 4.

If the bending samples height is less than 150 mm then 5% quantile of bending and tension strength shall be corrected up to the value corresponding to 150mm of the sample height, by dividing the result by:

$$k_h = \left(\frac{150}{h}\right)^{0,2} \tag{4}$$

where:

h – sample height in mm

Testing according to standard PN-77/D-04103 and PN-79/D-04102 does not meet employed assumptions of the testing scheme provided in standard EN 408 [4], and for this reason 5% strength quantile shall be corrected by dividing the result by:

$$k_l = \left(\frac{l_{es}}{l_{et}}\right)^{0,2} \tag{4}$$

where:

l_{es} or l_{et} = $l+5_{af}$

$a_f = 6h$

a_f, l – dimensions adequately normative and employed in testing

As presented in Table 3 (for comparison purposes) the results of characteristic strength in testing on small samples without defects, have also been presented without taking factor *ks* into account. Comparing Tables 3 and 4 great similarity of final results of the tested wood characteristic strength estimation has been noted. It may indicate that tests on small samples and their analyses according to EN 384 are reliable. However, interpretation of such a comparison shall be careful due to small number of tested models.

4. CONCLUSIONS

The subject of testing were 130-year-old timber beams obtained from floor construction of the monumental building, and the purpose of testing was their examination and checking their usefulness for reuse.

Based on the performed testing it can be stated that recycling of old floor beams from the demolished buildings is justified. Having no knowledge of the timber original parameters it is difficult to determine to what extent it reduced its strength. However, having tested the beams after the period of 130 years of their utilization, it was noted that strength parameters of the old wood are satisfactory and they can be reused. Another plus of old timber is the fact that that old beams, despite existing defects, are already dimensionally stabilized. The minus, however, is the need for additional cleaning of the beams and removal of damaged elements. Testing carried out on samples without defects of 20 × 20 mm section taken from earlier tested beams gave very promising results. Comparing strength of natural scale beams with strength of the little beams without defects determines in accordance with EN 384, great divergence of final results of the estimation of tested timber characteristic strength has been noted. For this reason testing of little samples and their analyses according to EN 384 has been reliable. However, the subject case needs to interpret this comparison carefully due to small number of the tested models.

Two damaged (destroyed) beams were repaired (strengthened) with CFRP during the testing in order to reconstruct their original load capacity and determine effectiveness of the existing old beams repair. The results of this testing will be the subject of separate paper.

When reusing timber elements after some demolition, very often decisive factor is profitability of such an activity. The basic factors which influence cost of obtaining wood from the structures demolished by the companies mediating its sale are cost of transport as well as costs related to cleaning, segregation and exposition of the obtained elements. Very often recycling of materials from demolition enables an investor to reduce cost of such a project as obtained material can be sold and cost of its utilization does not need to be incurred.

ACKNOWLEDGEMENTS

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Table 4. Strength properties of tested wood

	Mean strength $f_{śr}$ [MPa]	Standard deviation s [MPa]	Correction factor k_h [–]	Correction factor k_s [–]	Correction factor k_v [–]	Correction factor k_l [–]	Characteristic strength f_k [–]
Bending	65.44	10.49	1.50	–	1.0	1.32	24.36
Bending	65.44	10.49	1.50	0.77	1.0	1.32	18.76

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Abstract

In Poland, timber as a structural material is mainly associated with a historical building material of wooden cottages or sacred buildings in rural areas, as well as engineering facilities such as bridges or footbridges. In more contemporary applications, first of all timber is used for roof load-bearing structure (rafter framing in different static schemes and load carrying systems) and for timber floor load-bearing elements. In the past obtaining wood was not difficult due to quite large afforestation of Poland. However, the problems of the reuse of whole wooden elements built-in in the structures are practically unknown in our country.

Laboratory tests were carried out on 130-year-old wooden floor beams obtained from the monumental building under renovation. Original floor, whose beams were used for testing, was first made as beam-framed floor with sound boarding and ceiling. Testing elements delivered to the laboratory had got vivid restages of old floor – in the form of nails, sound boarding lathes and other impurities. Technical condition of the obtained beams was in the majority of cases good with visible longitudinal cracks. In some of the beams insects feeding well as biological corrosion marks were present.

The paper presents results of laboratory testing on timber beams after 130 years of utilization and evaluation of their technical parameters according to binding standard regulations. Additionally bending tests were carried out on little samples obtained from previously tested beams and compared to tests in natural scale.

As a results of the carried out laboratory tests it was preliminary noted that reclamation of old beams from demolished buildings is possible. If we do not know the original parameters of the timber it is hard to determine the level of strength loss. However, as a result of preliminary tests of full dimension beams obtained from 130 years old building it can be noted that strength parameters of old timber are good and they enable timber reuse. The advantage of old timber is also fact that old beams in spite of existing defects are dimensionally stabilized. The results of timber bending testing on little samples have been compared with results of testing of beams in natural scale. Having analysed the received results, great similarity of final results of estimated characteristic strength of the tested timber has been noted. It may indicate that testing on little samples and their analysis according to EN 384 is reliable. However, interpretation of such a comparison shall be careful due to small number of tested models.

Streszczenie

W Polsce, drewno to materiał konstrukcyjny kojarzony głównie jako historyczny budulec chat drewnianych lub obiektów sakralnych na terenach wiejskich, a także obiektów inżynierskich, takich jak mosty czy kładki dla pieszych. W bardziej współczesnych zastosowaniach drewno wykorzystywano przede wszystkim do wykonywania konstrukcji nośnej dachów (na więźby o różnych schematach statycznych i różnych układach nośnych) oraz jako elementy nośne stropów drewnianych. Dawniej z uwagi na stosunkowo duże zalesienie obszaru Polski, pozyskanie drewna dla inwestycji nie sprawiało trudności. Jednak zagadnienia ponownego użycia całych elementów drewnianych wbudowanych w wielu obiektach, są w naszym kraju praktycznie nieznane.

Do badań laboratoryjnych udało się pozyskać 130 letnie stropowe belki drewniane z remontowanego zabytkowego budynku. Pierwotny strop, z którego pozyskano belki do badań, wykonany był jako belkowy ze ślepym pułapem i polepą. Elementy badawcze dostarczone do laboratorium posiadały widoczne pozostałości po stropie – w postaci gwoździ, łat ślepego pułapu i inne zanieczyszczenia. Stan techniczny pozyskanych belek był w większości dobry z widocznymi podłużnymi spękaniami. W niektórych belkach były widoczne ślady żerowania owadów oraz widoczna korozja biologiczna.

W referacie przedstawiono badania laboratoryjne belek drewnianych po 130 latach użytkowania oraz ocenę ich parametrów technicznych według obowiązujących przepisów normowych. Wykonano również badania na zginanie dla małych próbek pozyskanych z przebadanych wcześniej belek oraz porównano do badań w skali naturalnej.

W wyniku przeprowadzonych badań wstępnie można stwierdzić, że odzyskiwanie starych belek z rozbieranych budynków jest możliwe. Nie znając parametrów pierwotnych trudno określić w jakim stopniu drewno obniżyło swoją wytrzymałość. Niemniej jednak w wyniku przeprowadzonych wstępnych badań na belkach pełnowymiarowych pozyskanych z 130 letniego obiektu stwierdzono, że parametry wytrzymałościowe drewna są dobre i pozwalają na powtórne wykorzystanie. Na korzyść drewna starego przemawia również fakt, iż belki stare mimo istniejących wad są już ustabilizowane wymiarowo. Porównano również wyniki badań drewna przy zginaniu na małych próbkach z badaniami belek w skali naturalnej. Po przeprowadzeniu analizy otrzymanych wyników stwierdzono dużą zbieżność końcowych wyników oszacowania wytrzymałości charakterystycznej badanego drewna. Co może wskazywać, iż wykonywanie badań na małych próbkach i ich analiza wg EN 384 jest miarodajna. Należy jednak ostrożnie interpretować to porównanie z uwagi na niewielką ilość badanych modeli.

„Badania naukowe zostały wykonane w ramach realizacji Projektu „Innowacyjne środki i efektywne metody poprawy bezpieczeństwa i trwałości obiektów budowlanych i infrastruktury transportowej w strategii zrównoważonego rozwoju" współfinansowanego przez Unię Europejską z Europejskiego Funduszu Rozwoju Regionalnego w ramach Programu Operacyjnego Innowacyjna Gospodarka."

Łukasz Hojdys[1], Piotr Krajewski[2]

Experimental tests on strengthened and unstrengthened masonry vault with backfill

Badania doświadczalne wzmocnionego i niewzmocnionego sklepienia z zasypką

Keywords: Strengthening, Vault, Arch, Masonry, Backfill, Composite Materials, FRG

Słowa kluczowe: wzmacnianie, sklepienia, łuki, konstrukcje murowe, zasypka, materiały kompozytowe, tynki zbrojone

1. INTRODUCTION

The one of the effects of the progress of civilization and society lifestyle changes is need for adaptation of existing buildings to a new function. Period houses in the market squares are converted into shopping centers, hotels, restaurants, pubs etc. Due to this conversion new loads for buildings must be considered and usually there is a need for strengthening of structural elements e.g. vaults [1, 2].

Nowadays externally-bonded composites are often used for vaults strengthening. Research performed in recent years indicated that fiber reinforced polymer systems [3-6] and systems that use cement-based matrices [7-9] are effective strengthening solutions for vaults. Most of these tests were performed on arches or vaults without backfill. In historic buildings buried vaults are common and it is obvious that vaults are not isolate structural elements but they interact with fill. Previous tests carried out on unstrengthened arches with fill material indicated that the presence of fill material and its properties strongly influence the behavior and load-carrying capacity of vault-soil system [10-13]. Most of these tests were performed on masonry arch bridge models but according to tests presented in [14] similar conclusions can be drawn in case of vaults in historic buildings.

Taking into consideration the facts mentioned above, the question arises whether strengthening systems based on composite materials are an appropriate solution for the vaults with backfill or not. The research presented in this paper was performed in order to observe collapse mechanism and determine load-carrying capacity of buried vault with and without strengthening. The masonry vault with light expanded clay aggregate backfill was tested twice. The element without strengthening was tested first and after application of the strengthening system it was tested again. Tests results were compared in order to check the effectiveness of applied strengthening system in the case of buried vaults.

2. CHARACTERIZATION OF TESTED ELEMENTS

The tested elements consisted of: unstrengthened or strengthened masonry vault supported by reinforced concrete abutments, fill material above the vault, end and side walls surrounding backfill material (Fig. 1, Fig. 2). The vault was built of clay bricks and lime mortar. Thickness, internal span and rise of the vault were 125 mm, 2000 mm and 730 mm respectively (Fig. 1). The width of the specimen was 1040 mm.

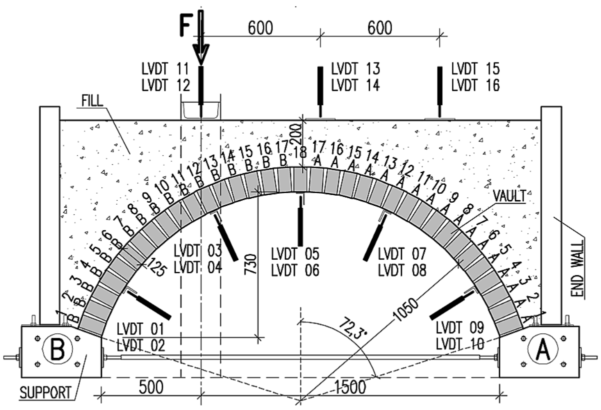

Fig. 1. Geometry of the vault, dimensions in mm. Arrangement of displacement transducers, brick courses numbering

The end walls were made of reinforced concrete whereas side walls were made of OSB or Plexiglas board stiffened with

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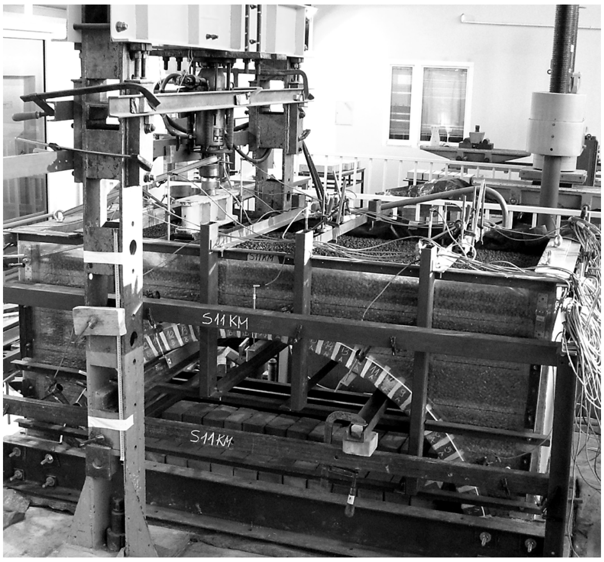


Fig. 2. General arrangement of tested vault – specimen S11KM during the test procedure

steel elements. The side walls were not structural elements so between walls and vault about 15 mm wide gaps were left.

In both tests light expanded clay aggregate was used as the fill material. The particle size ranged from 10 to 20 mm and bulk density was about 300 kg/m3. The fill material was placed and compacted in 200 mm thick layers. The total depth of the fill at the crown was equal to 200 mm. The vault in the second test was externally strengthened with alkali-resistant glass grid (Mepegrid G220) embedded in cement-based matrix (Planitop HDM). Strengthening system was supplied by MAPEI Polska Sp. z o.o. The grid is consisted of longitudinal (type I) and transversal (type II) fiber glass strands connected perpendicularly at about 25 mm spacing (Fig. 3a). In the research both components of strengthening system were tested – type I and type II strands of glass grid (according to PN-EN ISO 527-1) and grout specimens (according to PN-EN 12190). Selected mechanical properties of the matrix and glass fibers are presented in Table 1.

Table 1. Selected mechanical properties of strengthening system components

Material	Flexural strength		Compressive strength		Maximum tensile load*	
	MV (N/mm^2)	CV	MV (N/mm^2)	CV	MV (N)	CV
Planitop HDM after 14 ±1 days	10.5	19%	25.0	11%	–	–
Planitop HDM after 28 ±1 days	12.5	14%	31.8	12%	–	–
Mapegrid G220 "type I" fiber glass strand	–	–	–	–	1102	8%

* from "type I" specimens tests (see Fig. 3b-c)
MV – mean value
CV – coefficient of variation

Tested elements were loaded at a quarter span. The load was applied to the top of the fill material and was increasing continuously until failure. During the tests load, radial displacements of vault and vertical displacements of upper surface of the fill were measured and recorded. The arrangement of the load cell and displacement transducers is given in Fig. 1.

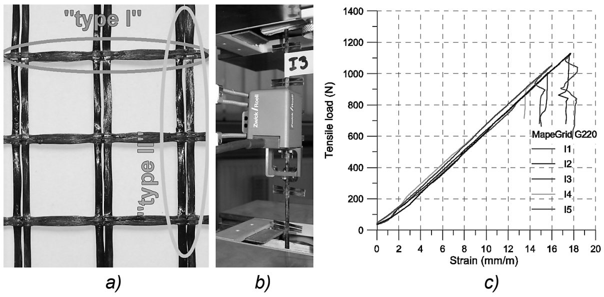


Fig. 3. Glass grid tensile tests: a) Mapegrid G220 glass grid – detail; b) "type I" glass grid specimen test, c) load-strain diagrams for "type I" fiber glass strands

3. TEST RESULTS

3.1. Unstrengthened vault – S11KM

The unstrengthened vault was the first tested element. During the test apart from measuring loads and displacements development of cracks were observed. The first crack appeared at a load of 7.7 kN between brick courses 13B and 14B. These brick courses were situated beneath loading point. The numbering of brick courses is given in Fig. 1. Next crack appeared between brick courses 12A and 13A at a load of 10.0 kN. At a load equal to 14.5 kN another crack became visible. It was situated between brick courses 13A and 14A. Next two cracks appeared at the extrados of the arch at a load of 16.7 kN. The first one was situated between brick courses 15A and 16A and the second one between brick courses 16A and 17A. At a load of 19.3 kN a new crack appeared at the intrados of the vault between brick courses 3A and 4A. Another two cracks became visible at a load of 21.5 kN. The first one appeared between brick courses 3B and 4B and the second one between brick courses 4B and 5B. At a load of 24.1 kN new cracks at the intrados of the vault were observed. They were situated between brick courses 4A and 5A and 5A and 6A. Finally at a load of 24.7 kN the tested element turned into four hinge collapse mechanism. The hinges P1, P2, P3 and P4 developed between brick courses number 13B and 14B, 15A and 16A, 6A and 7A, 4B and 5B respectively. The collapse mechanism is presented in Fig. 7a and photos of each hinge are given in

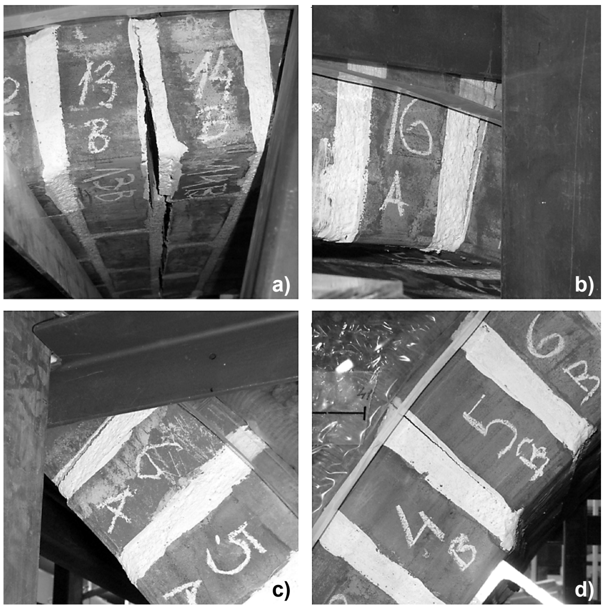


Fig. 4. Hinges of collapse mechanism for element S11KM (see Fig. 7a): a) hinge P1, b) hinge P2, c) hinge P3, d) hinge P4

Fig. 4. Mean displacements which were recorded at a failure load were given in Table 2. These displacement were calculated as an arithmetic mean of two recorded displacements from the transducers situated in the same cross-section of the vault.

Table 2. Displacements of element S11KM at failure load

Measuring point	01-02	03-04	05-06	07-08	09-10	11-12	13-14	15-16
Mean displacement (mm)	-0.74	-6.75	7.33	10.16	0.54	-9.22	11.87	11.68

3.2. Strengthened vault – S11W

After first test the vault without strengthening (S11KM) was prevented from collapse. The backfill was removed and the initial geometry of arch was restored. Then the strengthening was applied at the vault extrados. Alkali-resistant glass grid Mapegrid G220 was embedded in grout Planitop HDM (Fig. 5). The total average thickness of composite strengthening was about 7 mm. Strengthened element was cured for 14 days and then end and side walls were mounted and fill material was placed on the arch. Next displacement transducers and load cell were set in the analogical arrangement as during the first test.

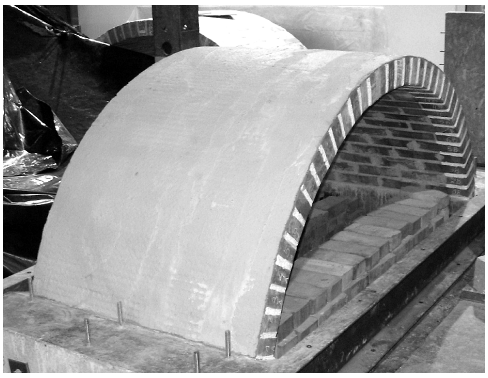
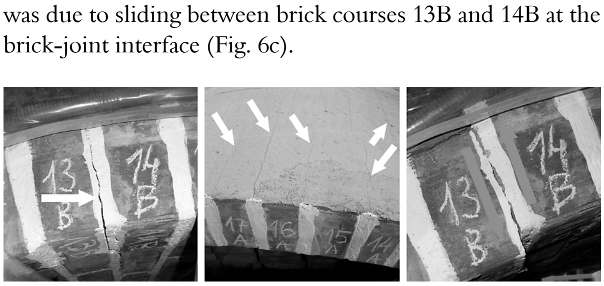


Fig. 5. Specimen S11W (strengthened vault)

During the second test cracks in joints (mainly at the brick-mortar interface) and in strengthening layer (mainly above joints) were observed. The first cracks appeared on intrados between brick courses 13B and 14B (Fig. 6a) and above support "A" at a load of 11 kN and 27 kN respectively. At a load of 41 kN cracks on the vault's extrados were observed. They were situated near the brick courses number 13A and 18 (Fig. 6b). At a load of 55 kN a sliding between brick courses 13B and 14B occurred. In a final stage of the test additional cracks were observed. They appeared above reinforced concrete supports "A" and "B". At a load of 79.95 kN significant increase of displacements at the loading point were observed and then the load started to decrease. The failure of the element S11W was due to sliding between brick courses 13B and 14B at the brick-joint interface (Fig. 6c).



a) b) c)

Fig. 6. Failure mechanisms of specimen S11W: a) first crack observed between 13B and 14B brick courses; b) cracks at extrados in strengthening layer observed after backfill removal; c) sliding along a mortar joint beneath the load application point.

4. DISCUSSION OF RESULTS AND CONCLUSIONS

According to test results presented above the load-carrying capacity of vault with strengthening was almost three times greater than for unstrengthened one. Precisely it was equal 24.69 kN and 70.95 kN for specimens S11KM and S11W respectively. The unstrengthened vault failed due to formation of classical four-hinge mechanism [15]. For the specimen S11W the failure was characterized by a shear sliding which occurred underneath the loading point along bed joint (Fig. 6c). The strengthening prevented joint opening at the extrados (except joints at abutments) and prevented failure due to four-hinge mechanism formation.

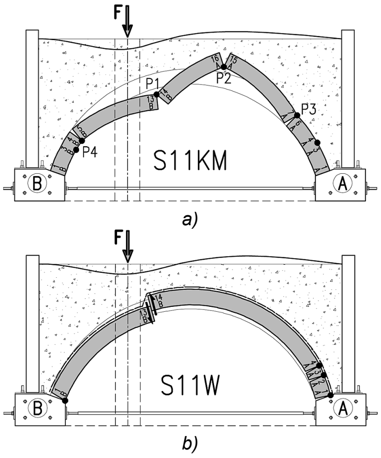


Fig. 7. a) Specimen S11KM – failure mechanism developed; b) Specimen S11W – failure mode observed

The comparison of the load displacement curves for considered vaults is shown in Fig. 8. The presence of the strengthening changed behavior of the buried vault not only in terms of a failure mechanism and load capacity but also in terms of ductility. Despite the fact that test of specimen S11W was interrupted just after reaching the maximum load, large deformation capacity of the strengthened vault prior to the failure is noticeable.

According to reports presented in the literature (i.e. [3] [6]) strengthening at the extrados of isolated masonry vaulted structures using composite systems is effective. Based on research results presented in this paper, glass grid embedded in cement-base matrix could be used as an efficient strengthening solution for buried vault.

It is worth mentioning that applying strengthening at the extrados of buried vault is favorable in the fire situation especially in case of FRP and TRM systems. Fill material placed above

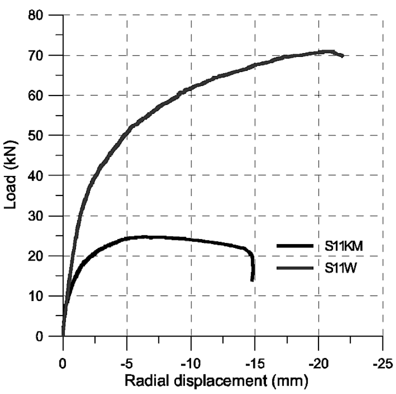


Fig. 8. Radial load-displacement diagrams for tested vault (S11KM – before strengthening; S11W strengthened specimen)

strengthening make additional thermal insulation. This is crucial when mechanical resistance (R) in the case of fire is required.

In many cases strengthening solutions based on composite materials could be more effective than traditional strengthening methods. Application of glass grids embedded in a cement-base grout allows to provide adequate load-carrying capacity of vault, reducing application costs and ensuring esthetic appearance.

ACKNOWLEDGEMENTS

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Abstract

This paper presents the results of two experimental tests on masonry barrel vault with fill material. The vault was built of clay bricks and lime mortar. Thickness, internal span and rise of the vaults were 125 mm, 2000 mm and 730 mm respectively. Light expanded clay aggregate was used as a fill material. The fill depth at the crown was 200 mm. The first test was performed on unstrengthened vault. In this case the main aims were to determine load-carrying capacity and examine the collapse mechanism of vault with backfill. In order to perform the second test the arch used in the first test was strengthened externally and tested again. The aims of this test was to determine load-carrying capacity and examine the general behavior of strengthened barrel vaults with fill material. Results of both tests were compared. The presence of strengthening influenced on load-carrying capacity and ductility of the vault. The strengthened element had higher failure load and was more ductile then vault without strengthening.

Streszczenie

W artykule przedstawiono wyniki badań eksperymentalnych sklepień z pachami wypełnionymi materiałem zasypowym. Elementy murowano z ceramicznej cegły pełnej na zaprawie wapiennej a zasypkę wykonano z keramzytu. Badania przeprowadzono na pasmach sklepień walcowych rozpiętości w świetle podpór wynoszącej 2000 mm, grubości 125 mm i strzałce 730 mm. Sklepienia obciążano w 1/4 rozpiętości aż zniszczenia rejestrując deformacje i poziom siły niszczącej. W pierwszym badaniu testowano sklepienie bez wzmocnienia wyznaczając jego nośność oraz identyfikując mechanizm zniszczenia. Po badaniu sklepienie wzmocniono powierzchniowo siatką z włókien szklanych i ponownie poddano testom. Celem badania było wyznaczenie obciążenia niszczącego oraz poznanie pracy sklepienia wzmocnionego z zasypką. Sklepienie wzmocnione charakteryzowało się większą nośnością i większą zdolnością do deformacji pod obciążeniem niż przed wzmocnieniem.

Andrzej Koss[1], Jan Marczak[2]

Evaluation of laser cleaning progress and quality

Ocena postępu i jakości metod czyszczenia laserowego

Keywords: Laser cleaning; Colourimetry; Laser spectroscopy; Artwork diagnostics

Słowa kluczowe: metoda laserowa, kolorometria, spektroskopia laserowa, ocena stanu dzieł sztuki

1. INTRODUCTION

Conventional methods of surface cleaning in conservation of art works are based mainly on mechanical or chemical techniques which are individually selected by experienced conservator. Cleaning of delicate objects, diverse from the point of view of materials composition needs not only extended expert appraisements of used substances, but also minimization of possible damages, always present in the case of mechanical cleaning. These traditional methods are very difficult to control. Chemical reagent shows similar interaction in conservation of paintings, where chemicals penetrate technological painting layers and causes permanent, difficult in analysis cross-sectional alterations. Conservation practice shows necessity of frequent treatments of sophisticated objects with complex technological structure and individual preservation state, resulting from influence of diverse external factors as well as changes in original building material itself. Every detail requires individual, predetermined cleaning parameters, which causes that application of conventional conservation methods is limited and difficult.

Laser technique gives possibility of almost full control of encrustation removal process at the surface of art works. Selective and precise interaction of light beam is fundamental advantage of non-invasive treatment of more or less tight unwanted surface layers. Specific properties of lasers, decreasing of systems costs, and reduction of dimensions of laser cleaning systems caused rising application of lasers in conservation [1-3]. Laser cleaning must be considered as an advanced tool applied in cases where traditional techniques may be inadequate. However, even if the last generation of laser systems has improved the comprehension of their effects and their engineering, laser cleaning is not yet a mature technology for earlier restoration tests, there is also lack of in-depth knowledge of the basic laser-artwork interaction mechanisms. There is still also a lack of diagnostic devices providing qualitative and quantitative information during the laser cleaning intervention.

Application of laser radiation in physico-chemical surface analyses and structural objects investigations, have started simultaneously with development of laser cleaning systems. In the face of increasing interest in laser cleaning and diagnostic systems, important is acquaintance of the conservation community with the fundamental advantages and shortcomings of laser radiation in treatment and analysis of matter.

In the present paper, authors describe and discuss selected areas of applications of lasers and optoelectronics in conservation of monuments and works of art, with particular attention paid to noninvasive, physico-chemical and structural analytical methods. Presented data are based on almost twenty years experience in laser cleaning and diagnostics of dozens priceless objects in Poland and abroad [4-6].

2. ENVIRONMENTAL POLLUTION AND CLEANING

Increasing pollution levels of monuments and sculptures exhibited in the open air inside built-up areas are the result of environmental pollution, composed mainly of soot and dust emitted by industrial objects or rising into the air from the earth surface. Next pollution groups originate from motor exhaust gases and substances generated by modern industry. Their influence on environment, including biological effects, is by far stronger than in the case of particulates. Effective detection, counteraction and removal are respectively much more difficult, in some cases unknown are efficacious restoration procedures.

Fast degradation of human natural environment lasts over 200 years, but main changes were the result of last seventy years of neglected control of industrial development. The percentages of artwork surface associated with corrosion damage (see e.g. Figure 1) reaches yearly 3 to 5% in dependence on monument localization and atmosphere pollution [7]. The complex nature of the formed unwanted superficial layers and limited versatility of any diagnostic

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method requires a combination of several surface analytical techniques for the complete characterization of deposits. This procedure should always be developed prior to the conservation treatment.

a) b)

Fig. 1. a) Pair of bronze putti with gilding at the garden's façade of the Wilanów Palace in Warsaw. Black arrows indicate microphotographs of damage details; b) Limestone figure of St. Vlah, patron of Dubrovnik, Croatia. Left side of figure shows the result of laser cleaning

Beside the artistic and aesthetic depletion of the artwork, deposits and encrustation can cause further degradation processes of both a physical and a chemical nature, requiring prompt restoration interventions before the artistic content of the stonework is irremediably lost [8]. Such an active conservation include surface cleaning which is often one of the first actions to be undertaken. It represents a crucial step in the whole restoration procedure, as the effects of this operation are irreversible and influence the future conservation of the restored artworks. Cleaning itself is also an important part of the artwork stabilizing process and is one of the most important processes in the active conservation of artifacts, preparing possible further treatments if needed: consolidation, coating of a surface or reconstruction of totally damaged elements.

3. PHYSICO-CHEMICAL AND STRUCTURAL DIAGNOSTIC METHODS

The main aim of analysis of monuments and works of art is identification of artwork structure, its important chemical components as well as characterization of its preservation state, including determination of influence of external factors. It is well known, that nature of a given conservation problem prescribes future utilization of specified technology or a combination of several techniques. Therefore, many powerful analytical methods developed as a result of technological progress in optoelectronics are addressed now to solve different and complex problems, arising in art conservation. It applies to different conservation studies, from identification of substrate and top layer materials (pigments, painting media, varnishes), through mapping of structural defects and mechanical discontinuities, up to dating, authenticity studies and careful removal of unwanted layers (encrustation, overpainting, old varnish etc.). Obviously, the most safe for objects are noninvasive techniques *in situ* (if possible to apply).

Main laser and optoelectronic techniques involved in physic-chemical and structural studies of artworks are following:

- Object chemical structure and surface analysis:
  - reflection/scattering;
  - absorption/transmission;
  - laser spectroscopy: induced fluorescence (LIF), induced breakdown (LIBS),Raman;
- Object physical structure and defects localization:
  - multispectral imaging;
  - interferometry, speckle interferometry;
  - holography, holographic interferometry;
  - laser vibrometry;
  - thermography;
  - laser tomography (OCT);
- Object morphology:
  - 3D laser scanning;
  - triangular scanning;
  - surface profilometry (mechanical, laser);
  - laser scatterometry;
  - laser tomography (OCT).

Particularly popular analytical methods have become spectroscopic techniques (laser and non-coherent), mainly due to their sensitivity, flexibility and analytical methodology [9]. Spectroscopy delivers information, which is directly or indirectly connected with chemical nature of investigated materials. Wide application in the diagnostics of historical object found classical Fourier infrared spectrometry (FTIR) or its DRIFT variety with the utilization of diffuse reflection of radiation. Range of FTIR spectroscopy applications include:

- identification of molecular compounds created at the artwork surface,
- studies of composition of painting layers,
- identification of fibers material, chemical composition and soiling of paper and parchment,
- investigations of epoxy resins.

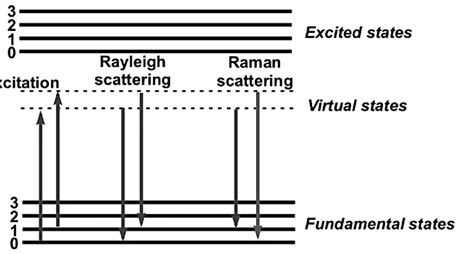
Optical measurement methods (scaterrometry, shadowgraphy, microscopy, reflectometry) are commonly supplementing sets of diagnostic methods. Increases interest in application of multispectral imaging for evaluation the results of laser cleaning, identification and mapping of painting materials and visualization of top surface layers.

Diagnostic techniques that utilize X-ray radiation and methods of nuclear physics and chemistry are also supporting conservation of artworks. The most popular is scanning electron microscopy (SEM), frequently with radiation energy dispersion (EDS or EDX). Chemical and crystallographic surface modifications, composition and volume structure of pigments and other materials are studied with the use of X-ray diffraction (XRD) and fluorescence (XRF). Additional basic materials research is sometimes realized using complex systems of mass spectrometry and atomic force microscopy.

4. LASER BASED DIAGNOSTIC METHODS

4.1 Raman spectroscopy

Raman spectroscopy is a light scattering technique, and can be thought of in its simplest form as a process where a photon of light interacts with a sample to produce scattered radiation of different wavelengths (Figure 2a). Molecular Raman spectroscopy is particularly well-suited technique for the determination of the artistic materials in a non-destructive way. By focusing a low power laser on a sample the intensity of the inelastically scattered light is plotted against the Raman



a)

b)

Fig. 2. a) Schematic illustration of energy levels involved in Raman phenomenon; b) R-3000 Raman system head during tests of medieval icon

wavenumber, which is proportional to the difference in energy between the laser and the scattered light.

Identification material tests and preliminary studies of deposits were conducted using two small Raman spectrometers, namely R-2001 from Ocean Optics, Inc., and R-3000 from Raman Systems, Inc. (Figure 2b). Raman spectra of superficial layers before and after laser cleaning were registered using a dispersive Nicolet Almega Raman spectrometer, equipped with two laser illumination sources (532 nm and 780 nm), Olympus BX research-grade confocal microscope (magnification 50×, long FD) and high precision motorized x-y stage. Nominal laser power of 25 mW (532 nm) and 35 mW (780 nm) was usually reduced (to around 30%) due to the small diameter of the focused laser spot (~2 μm) and sensitivity of the samples. Exposition time was 15-60 s in each of the two averaged scans of the sampled areas. Species identification followed application of the Thermo Scientific OMNIC Specta software.

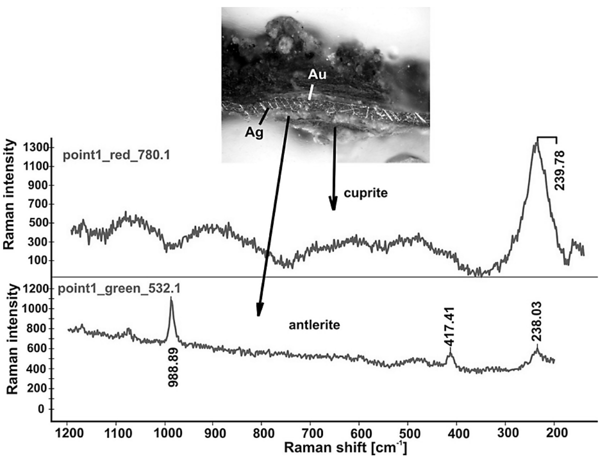


Fig. 3. Raman spectra of corrosion layers near the putto gilding. Small photograph shows the section of gilding. Identified metals and compounds are indicated by lines and arrows

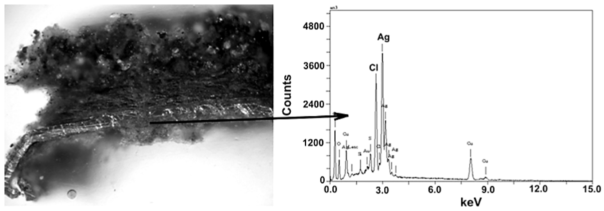


Fig. 4. a) Section of putto gilding damage with developed area of grey corrosion near Ag layer; b) SEM EDS spectrum of corrosion in the point indicated by black arrow and identified as cerargyrite (AgCl)

Application of Raman spectrometry allowed for the identification of several corrosion compounds on metal artworks and specimens. Two examples are shown in Figures 3-5.

Analysis of gilding sections taken from putto with laurel (Figure 1a) is presented in Figures 3 and 4. The pair of gilded bronze putti are dated to the end of the 17th century and attributed by annotations to the Rome studio of the Dutch sculptor Disqenue. Their function as decorative elements of the Wilanów Palace in Warsaw (garden façade) has caused serious soil, local corrosion of the alloy under the gilding as well as numerous mechanical damages. Raman spectra of grren salts, registered at section shown on a small photograph in Figure 3 have been identified as antlerite $Cu_3SO_4(OH)_4$ (Figure 3: upper plot), and can be found below and above the metallic layers.. Cuprite Cu_2O has been identified as the red compound indicated below green layer of antlerite. Grey salts, shown in Figure 4 (left photograph) which were not identified in Raman spectra, are probably silver compounds, as indicated by supplementary SEM EDS analysis (right side spectrum in Figure 4).

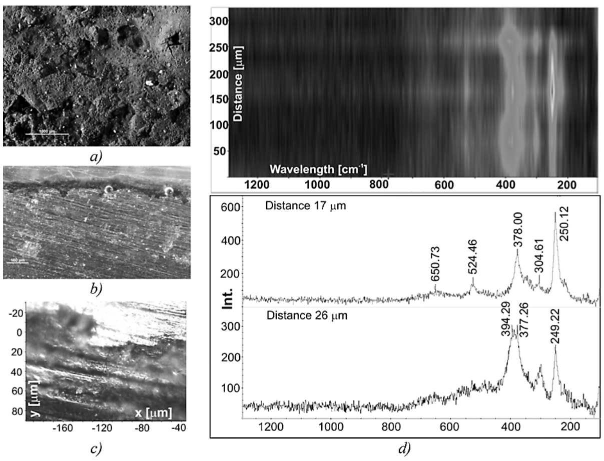
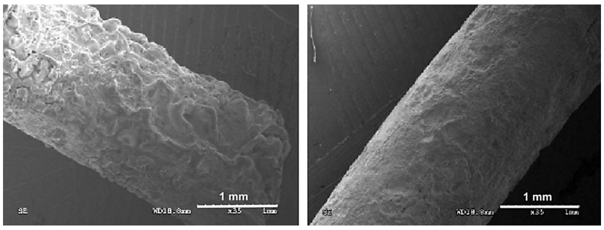


Fig. 5. Raman analysis of corrosion products on carbon steel specimen after annual outdoor exposition in the Railway Museum in Warsaw: a) photograph of surface; b) section of corrosion layers; c) microphotograph from Raman microscope indicating measurement line; d) Raman spectra image (upper photo) and graph below

The second example – analysis of a carbon steel sample after annual outdoor corrosion test in the Railway Museum in Warsaw is presented in Figure 5. Two main compounds were identified: lepidocrocite (red line 378 cm^{-1} and 250 cm^{-1}) and goethite (blue line, 378 cm^{-1} and 298 cm^{-1}).

The laser-deposit interaction in the long laser pulse range resulted in melting, evaporation and sputtering of the surface layer.

Interesting results have been obtained during tests of the influence of laser pulse duration on the quality of post-processing surface of artwork [10]. It is common knowledge that the interaction of long laser pulses (dozens of microseconds) with a solid in a free running regime of Nd:YAG laser



a) *b)*

Fig. 6. SEM photographs of fragments of archaeological bow from the collection of the Wilanów Palace Museum in Warsaw after laser cleaning: a) laser pulse duration 150 ms; b) laser pulse duration 120 ns

causes the generation of intense heat on the worked surface. However, the melting and mixing of layers is so efficient that the Raman identification of the resulting surface composition is not possible due to the overlapping of many Raman peaks. An example of such strong influence of laser induced surface heating and melting of archaeological bronze bow is shown in Figure 6a. At the other side, interaction of short, nanosecond pulses in the Q-switched generation regime of Nd:YAG laser resulted in selective, step by step removal of corrosion layers, without damage of the underlying substrate (Figure 6b). The final results of Raman analysis of bow are following:

Corroded surface before cleaning consists of cuprite Cu_2O, antlerite $Cu_3(SO_4)(OH)_4$, malachite $Cu_2CO_3(OH)_2$ and posnjakite $Cu_4(SO_4)(OH)_6H_2O$;

Surface after cleaning with long microsecond laser pulses contain a mixture of different copper compounds with metallic copper and tin (from the original bow substrate)

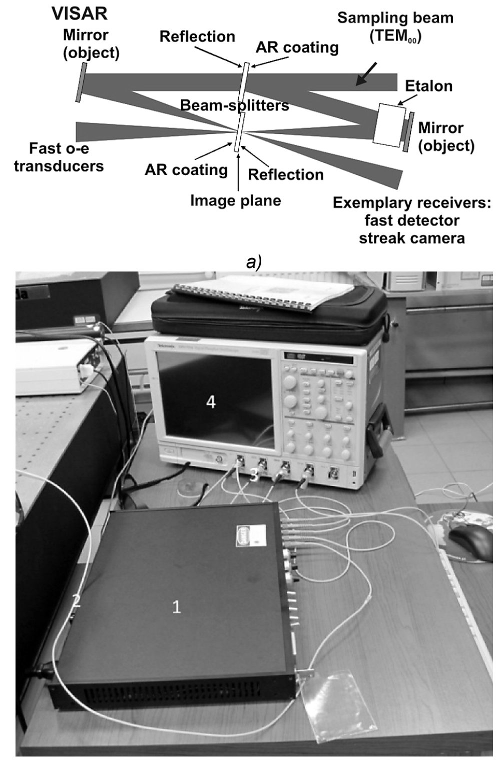
Surface after cleaning with short nanosecond laser pulses shows preserved original substrate and patina, consisting of not touched cuprite layer and thin layer of antlerite.

4.2. Laser interferometry

Laser interferometry is a well-established, highly sensitive technique for non-destructive testing and analysis. It can be useful in the field of historical object conservation in reconstruction of 3D structure of the object under observation and its deformation under stress, layer-to-layer detachments, delaminations and surface cracks [11]. As it is shown in Figures 5-6, sensitive laser interferometry and its variant – Doppler VISAR can be also utilized for "on-line" analyses of shock waves generated during laser cleaning of quite thick objects. As the amplitudes of registered shock wave pulse decrease with the decay of soiling layer, it can serve as an indicator of laser cleaning level. At the other side, substantial signal increase may warn against possible hidden defect in the structure of fragile object and stop the laser. This technique may be useful particularly during the "on-line" diagnostics of cleaning of thin and fragile objects.

In our first experiments, samples of stone materials (granite, marble and sandstone) were specially prepared for interferometric studying of generation of shock waves during interaction of high intensity pulse laser radiation [12]. Recently, a new approach has been introduced, based on optical fiber Doppler velocity interferometer (VISAR) [13], developed by Martin Froeschner and Associates, USA. The example of measurement scheme and overall view of the system are presented in Figure 5.

Optical backscattered signals can be registered from the front or back side of the object under treatment. Acquired information includes surface movement amplitude and velocity data in the time domain scale. Figure 6 shows examples of oscillograms registered for 4 mm thick marble samples irradiated with Nd;YAG Q-switched laser (1064 nm).



b)

Fig. 7. a) Optical scheme of Doppler velocity interferometer (VISAR); b) Photograph of model FDVI Mark IV-3000 system: 1 – main interferometer body, 2 – signal optical fiber, 3 – digital scope

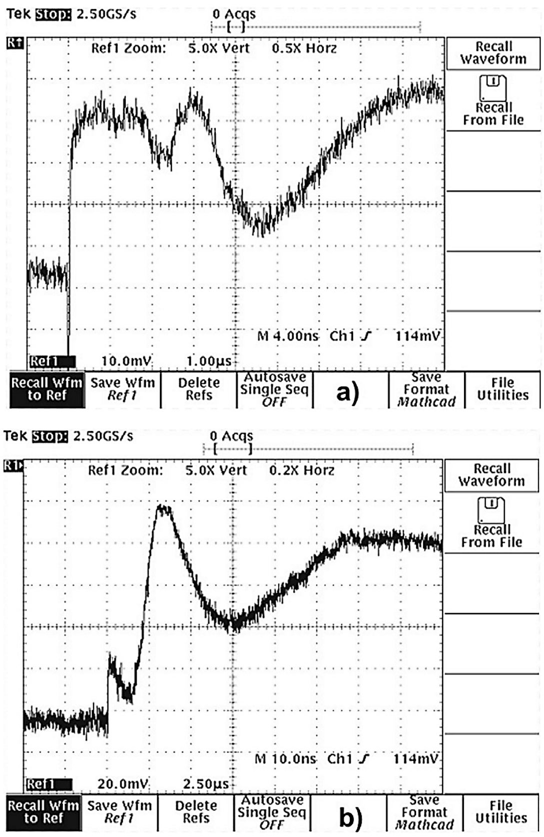
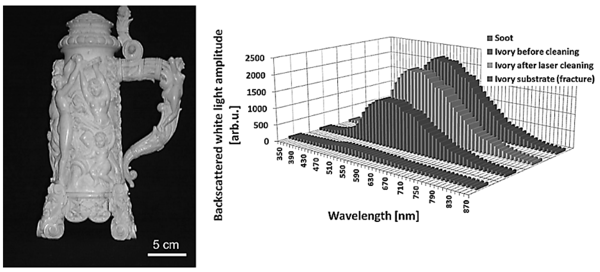


Fig. 8. Oscillograms of shock-wave generated in stone samples by intense pulse laser irradiation: a) fluence 2.5 J/cm2; b) fluence 4.5 J/cm2. Note change of vertical scale in b)

5. MEASUREMENT OF LIGHT DIFFUSION REFLECTION COEFFICIENT AND COLOURIMETRY

Physical parameter which allows description of inhomogeneous encrustation optical characteristics is average reflection coefficient of backscattered white light (or laser light). Its modified version – spectrometric measurement of amplitude of diffusively reflected white light in the function of wavelength represents synonymous and objective colourimetry. Apart from easy discrimination between clean and contaminated surface of artwork, the results of spectrometric measurement of reflection coefficient are useful in matching laser radiation wavelength to the highest absorption coefficient of layers to be removed.

Our sophisticated measurement system, developed for precise measurements of spectral reflectivity to determine laser cleaning level and optimise processing wavelength of radiation was described elsewhere [14]. Figure 9b shows the results of measurements of light amplitude scattered from an ivory artwork (photograph in Figure 9a).



a) b)

Fig. 9. a) Ivory mug (original view before cleaning); b) Spectral intensity of diffusely reflected white light [14]

Spectral reflectance data can also express colors ranking by means of tone (hue), clarity (brightness) and saturation (chromaticity). Determination of their scale creates possibility of objective digital and convenient color measurement. Different data representation are calculated, from which the colour space L*a*b*, determined also as CIELab, is now one of the best known and wide used in almost all domains for object colour measurements. Also the well known empirical test of laser cleaning efficiency relies on comparison of object color in dependence on laser fluence [15]. L* describes the brightness, a* the red-green color and b* the yellow-blue color in CIELab colour space. Additional attribute of colourimetric measurement is documentary notation, which determines reference point and will allow to return to same hue after few dozens of years during next renovation procedures.

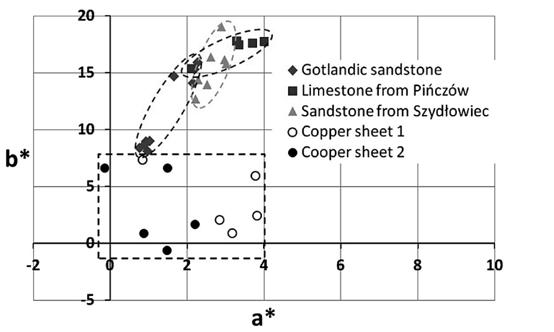


Fig. 10. Set of colourimetric experimental data b = f(a*) for laser cleaning of different materials*

Figure 10 contains the image of colorimetric data obtained with the Konica Minolta CM2600d spectrophotometer in the four experiments: laser cleaning tests of Gotlandic sandstone sample, in two places of throne wall of the King's Batory Chapel in Wawel Cathedral, Cracow [15], and for two regimes of laser cleaning of old copper sheets, using 6,2 ns (cooper sheet 1) and 8.4 ns (copper sheet 2) laser pulses. Charts show two important analytical conclusions. The first indicates arrangement of stone measurement points for stones along a characteristic line (hue angle = arctan (b*/a*), which is similar for sandstones (Gotlandic, from Myślenice) and quite different for limestone from Pińczów. In case of cleaning of copper sheets, large variations of colourimetric coordinates a* and b* (points in rectangular dashed area) are connected with transition of laser cleaning beam between individual layers of deposit and corrosion – from black crust to brochantite (increase of green component i a* and blue in b*), and from brochantite to cuprite (decrease of a* and b*) [16]. It can be easily concluded that colourimetric analysis of laser cleaning process should be limited to the homogeneous superficial layers on artworks and monuments.

6. CONCLUSIONS

Discussion presented in this paper is limited to the selected diagnostic methods. Authors extensively investigated also LIBS [17], acoustic effect of laser-matter interaction [18], and Optical Coherence Tomography (OCT) [19], but analysis of all other methods exceeds the available volume of the present paper. Summarizing, the paper presents our modest attempt to describe and discuss analytical techniques applied to mainly to characterize laser cleaning – comfortable and effective technique for the process of encrustation removal. Raman microscopy is the ideal technique for the investigation of materials used on works of art because it is reliable, sensitive, specific, nondestructive and can be applied *in situ*, therefore avoiding any sampling and consequently any damage to the object under examination. Interferometric systems (VISAR) seem to be present and future advanced technique particularly useful in the basic research in the field of laser – art matter interaction. Colorimetric representation of reflectance data in the CIE-L*a*b* color space is widely utilized to study several conservation problems, including physical-chemical modifications of the surface induced by cleaning (laser irradiation), effectiveness of cleaning procedure, artwork ageing process as well as serves as a documentation of artwork surface digital color representation for future restoration works.

ACKNOWLEDGEMENTS

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Abstract

The estimation and analysis of damages (present condition), object conservation (cleaning process), and the protection of an object against further degradation are the main tasks of conservator. One of the physical methods that is becoming more and more popular for unwanted deposit removal is the laser cleaning method. Laser tool is non-contact, selective, local, controlled, self-limiting, gives immediate feedback and preserves even the gentlest of relief – the trace of a paintbrush. Paper presents application of different, selected physical methods to characterize condition of works of art as well as laser cleaning process itself. It includes, tested in our laboratories, optical surface measurements (e.g. colourimetry, interferometry), thermography, and acoustic measurements for "on-line" evaluation of cleaning progress. Results of laser Raman spectrometry analyses will illustrate identification of objects superficial layers.

Streszczenie

Ocena i analiza uszkodzeń (stanu aktualnego) oraz obiektu poddanego konserwacji (np. w procesie czyszczenia) i zabezpieczenie obiektu przed dalszą degradacją stanowią główne zadania konserwatora. Jedną z metod fizycznych, którą staje się coraz powszechniej w usuwaniu niepożądanych nawarstwień, jest metoda z wykorzystaniem promieniowania laserowego (metoda laserowa). Laser, jako narzędzie konserwatorskie jest: bezkontaktowe, selektywne, lokalnie kontrolowane, samo się ograniczające, dające natychmiastowe sprzężenie zwrotne i zachowujące nawet najdelikatniejszy relief – zachowujące ślady pędzla artysty malarza. W artykule przedstawia się zastosowanie różnych, wybranych metod fizycznych, aby scharakteryzować aktualny stan dzieła sztuki, jak również sam proces czyszczenia laserowego. Artykuł zawiera opis przeprowadzonych testów w laboratoriach, pomiary optyczne powierzchni (np. kolorymetryczne, interferometryczne), termowizyjne i akustyczne, w celu oceny procesu czyszczenia laserowego „on line". Przedstawiono ponadto przykłady analizy warstw wierzchnich z wykorzystaniem spektrometru Ramana.

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19th century curved board roofs in Bavaria

XIX-wieczne dachy krążynowe w Bawarii

Keywords: Timber structures, Curved board roof, Construction process

Słowa kluczowe: konstrukcje drewniane, dachy krążynowe, proces konstrukcyjny

1. INTRODUCTION

Roofs relying on laminated arched trusses are one of the most idiosyncraic construction types of 19th century timber building. This kind of structure never got widespread, but nevertheless, it received a lot of attention and discussion in the contemporary literature and re-appeared every now and then in the built reality.

New building types such as railway stations, riding halls, or theatres frequently required wide-span roofs; together with the dwindling supply of timber and the influence of neo-classical architecture (calling for low-pitched roofs), this forced German architects and carpenters to move away from the classical „liegender Stuhl" constructions (having a reputation of being extremely timber-consuming) and to try purlin roofs. It was hoped that new construction types would require less timber and would therefore save money and reduce the risk of conflagration.The experiments in structure are particularly well reflected by the development of the curved board roof.

2. 19th CENTURY DEVELOPMENT OF THE CURVED BOARD ROOF TRUSS

The early history of the curved board roof around 1800 has been traced very well in [1]. Our present study tries a tentative extension of this work until the late 19th century. It is based on the extensive contemporary printed sources on the subject, as well as a survey of several existing buildings employing this roof type.

The 19th century technical literature proves that curved board roof trusses are a recurring subject until the end of the century. Typically, Philibert de l'Orme is cited with his book "Nouvelles inventions pour bien bastir et á petis fraiz" of 1561as the inventor of the laminated arched roof truss [2]. In Germany, and particularly in Prussia, this standard historical genealogy is primarily due to David Gilly, who launchend a big propaganda in favour of arched roof trusses around 1800 [3]. It is not the topic of the present study to find out whether Gilly's etymology was correct or not. Anyhow, it should be noted that arches made up of several layers of curved boards – set on edge and nailed together – are the principal construction of 16th-19th century wooden vaults, so that not "re-invention" was really necessary to bring the idea back to the constructional practice of Gilly's time.

In the early 19th century, we find a diversity of different constructions in the general category of curved board arched trusses. David Gilly himself has left a record of constant experimentation and improvements in his various treatises. His first curved board roofs employed pairs of arched rafters which were notched into a board on edge forming a kind of „ridge purlin" (Fig. 1, no. 93 A). However, observed damages in this kind of structure led him to an improved version: Now, he preferred rafters halved across each other at the apex and resting on top of the purlin (Fig. 1, no. 93 B and C). This was also presumed to improve the longitudinal stiffness of the roof ([1], p. 53).

In 1801, the Saxonian master carpenter Leopold Leideritz dealt at length with Gilly's roofs and published his thoughts in his textbook on carpentry. His main objection against Gilly's constructions was the use of short boards. Leideritz recognised correctly that rafters made up of short pieces joined together were considerably less stiff than rafters consisting of as few pieces as possible ([5], p. 173). This was an essential argument against both Gilly and his purported "ancestor" de l'Orme because both of them had praised the cheapness of the curved board arches which could be made up of short, otherwise "useless" pieces and would not require large scantlings. Furthermore, Leideritz criticized Gilly's ridge "purlin". He considered it as insufficient for the required longitudinal

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stiffening of the roof. Leideritz recommended, to join the individual arched trusses in ridge direction by horizontal braces at half-elevation (mortise-and-tenon connection). In the last volume of his textbook, which appeared only in 1818, Leideritz added further criticizm. This time, his argument was based on his own experience with an arched roof erected on a barn ([6], p. 214). The experience confirmed him in his opinion that Gilly's original designs were defective.

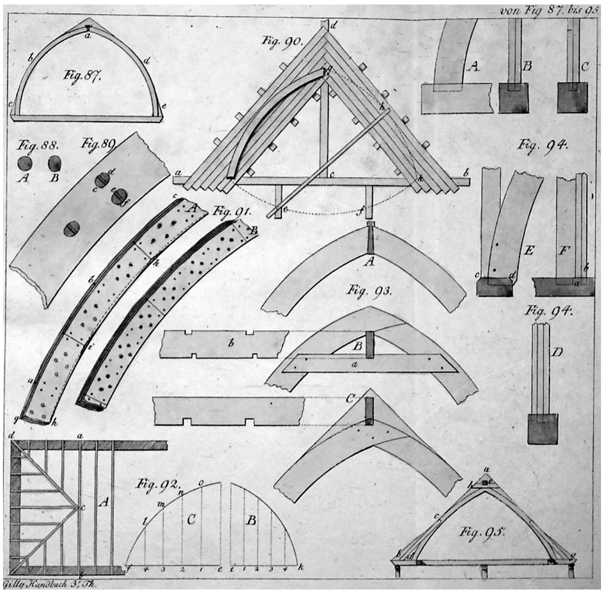

Fig. 1. David Gilly's arched roof truss designs [4]

Gilly himself could not entirely repel these arguments. He himself added more longitudinal members to his roof designs in his publications after 1800. In his riding hall erected at Berlin-Kreuzberg (erected in 1799/1800, see [1], p.217; cf. Fig. 2), Gilly used the laminated arches as the load-carrying truss for an ordinary roof with straight rafters. While this roof did not differentiate between "principal" and "ordinary" rafters, but consisted of a sequence of identical rafter pairs, longitudinal connection was provided by a pair of purlin-like beams resting on the collar beam. At the rafter feet, additional inclined struts were added; these also clasped a purlin-like longitudinal member. In addition to this, Gilly added more wind bracing in the plane of the rafters. Note, however, that this roof did not contain a ridge purlin. Friderici – the editor of Gilly's posthumously published third volume of the "Landbaukunst" from which our illustration is taken – emphasized that collar beams, braces and rafters were "intimately joined" in this roof ([4], p.175, §91).

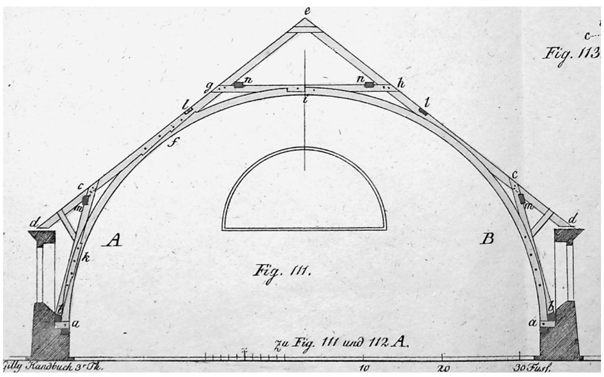

Fig. 2. Berlin-Kreuzberg, riding hall of the „Leibhusaren-Regiments" [4]

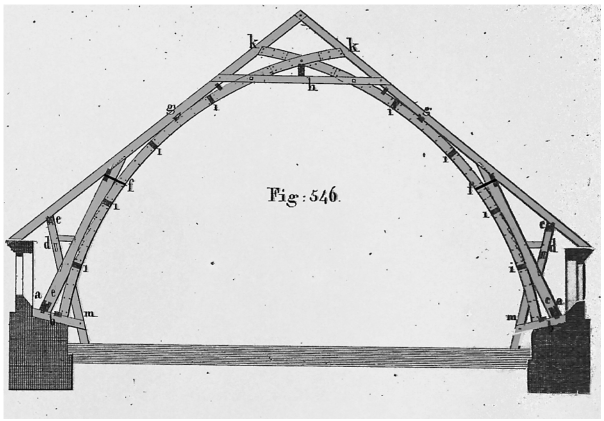

Fig. 3. Riding hall at Berlin, Husarenstraße. [7] pl. LXXIII, Fig. 546

A later example, the riding hall at Husarenstraße in Berlin (according to [1], p.64, this was probably erected in 1818, cf. Fig. 3), shows a similar development of Gilly's original scheme. In this roof, the arches of two neighbouring trusses are connected to each other by a whole set of horizontal braces at regular intervals. The inclined struts are bolted down to the arches ([7], p. 61). At the apex, the pair of arched rafters cross each other and abut against the straight rafters forming the outer shape of the roof. The straight rafters – originally only introduced in order to eliminate the difficulty of getting a tight cover on a curved roof surface – are thus increasingly integrated into the load-bearing structure of the roof truss. Originally, Gilly had added short straight extensions on top of the principal curved rafters for the purpose of flat roof surfaces; now, the rafter is an integral part of the scheme.

The time's tendency towards low roof pitches inspired another innovation: Since the arch requires considerable height in order to work properly, the outer shape of the building can only be adjusted to the neo-classical idea of a low roof pitch if the eaves are raised above the rafter feet (cf. [1], p. 59). Another important roof with arched trusses built at roughly the same time is the roof over the central pavilion of the Polytechnic of Vienna (Fig. 4, erected 1816-18).

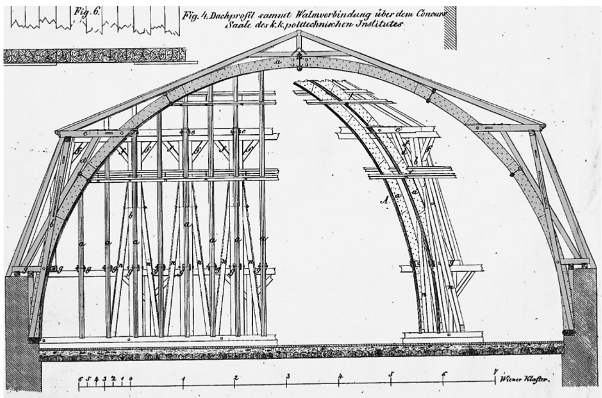

Fig.4. Vienna, Polytechnical School, roof over the main pavilion ("Concours-Saal"). [8] Fig. 4, pl. XXII

This roof, construction details of which were published by Johann Gierth in his 1840 textbook ([8], p. 306), has been preserved and recently restored. In this roof, the individual arches are joined by pairs of strong boards which clasp the arches from above and below. Furthermore, inclined wind-bracing was added to each arched truss. The roof, a mansard-shaped pavilion roof, also has a crippled collar beam which joins the arched rafter in its middle to the outer rafters. The collar beams are supported by a purlin resting on an inclined strut, a construction which is highly reminiscent of the traditional „liegender Stuhl" . The inclined strut is present only in every second rafter pair, so that this roof is one of the earliest arched roofs which differentiates between „principal" and „ordinary" rafters.

Further development was brought about for the arched board roofs by Karl Friedrich Schinkel, who had studied at Gilly's „Bauakademie". The roof over Schinkel's Berlin „Schauspielhaus am Gendarmenmarkt" (erected 1818-22) combined all the improvements and innovations of the preceding two decades (Fig. 5). It is evident that the curved board roof now tends towards the structure of a fully developed purlin roof.

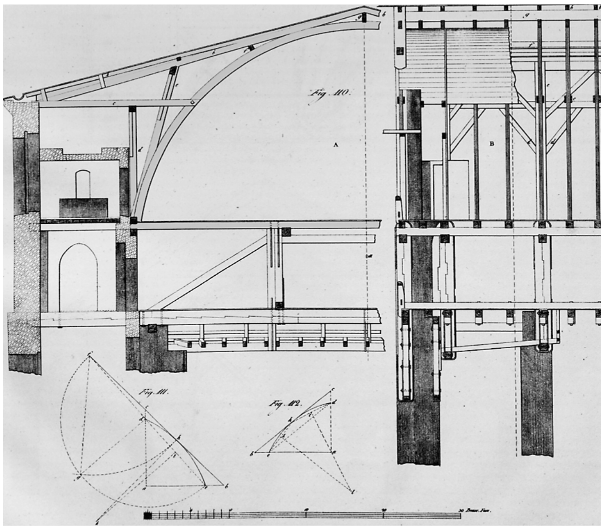

Fig. 5. Berlin, Schauspielhaus, roof over the painters' hall. [12] pl. XII, Fig. 110

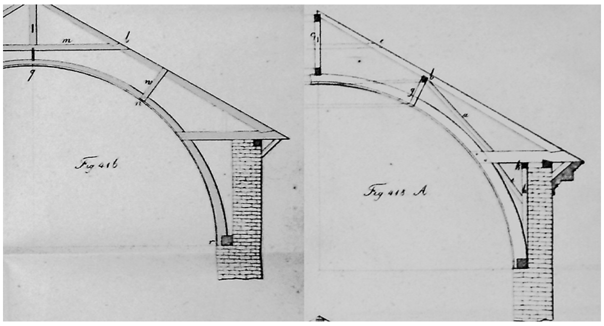

Fig. 6. and Fig. 7. [9] pl. XXIII

France did not take part in Gilly's "curved plank roof" frenzy; France had its own version of that, namely the "laminated arched roof" developed by the colonel Amand-Rose Émy. Émy's arches consisted of layers of flat boards placed on top of each other and bent into arched shape. His scheme was employed in the construction of several military halls in France, staring in the 1820ies. Émy's arched trusses are beyond the scope of the present contribution. However, it is important to note that the scheme was also well-known to German engineers, albeit from publications rather than from built examples (publications included a German translation of Émy's 1837-41 textbook on carpentry). Some influence may have been exerted by this French counterpart. We are particularly inclined to believe that the increasing use of radial braces clasping the arch were inspired by the French construction. Examples of Gilly-type roof trusses with radial braces include two roof designs as published by Ludwig Friedrich Wolfram in 1824, cf. [9] (Fig. 6 and Fig. 7).

Perhaps even closer in structure to Émy's arched trusses is a later roof designed by Schinkel, his riding hall erected for Prince Albrecht (1831, Fig. 8). This Gothic Revival structure is covered by means of pointed plank arches, joined to the outer straight rafters by Émy's radial struts.

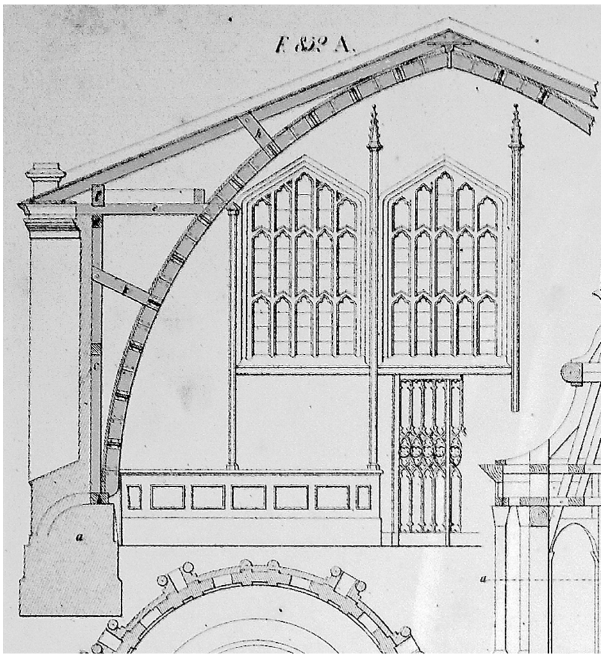

Fig. 8. Berlin, Prince Albrecht's riding hall. [10] Fig. 852, pl. 126

It took a while before the curved plank roof, originating at Berlin, finally arrived in Bavaria as well. Publications such as Johann Michael Voit's treatise of 1825 may have contributed [11]. Voit had been Gilly's student and was infected by Gilly's ideas. When he became a civil servant at Augsburg, he spread the innovative ideas there and believed in the general superiority of arched girders. In contrast to his contemporaries who tended to make the arches semi-circular rather than pointed except in „Gothic" style buildings, Voit remained faithful to the pointed shape as favoured by Gilly himself (Fig. 9).

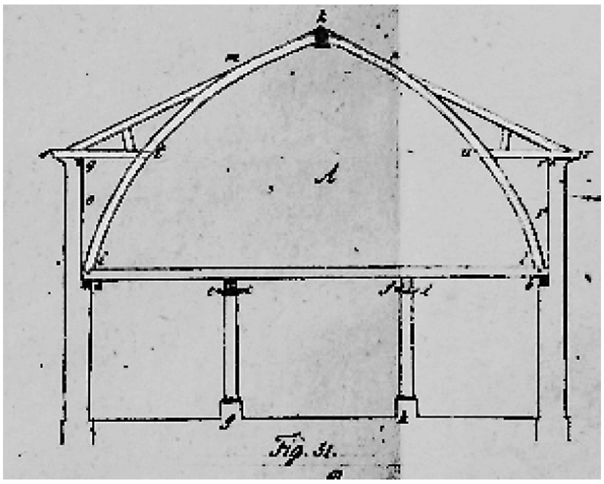

Fig. 9. recommended arched roof for storage buildings and packhouses. [11] Fig. 51

Curved board roof structures were also contained in the most influential German textbook on carpentry, namely, Johann Andreas Romberg's book of 1833 [7] (an extended second edition appeared in 1847). Romberg published, in addition to Gilly's original designs, the roof construction of Schinkel's „Schauspielhaus". The fact that the second edition of Romberg's work was dedicated to Leo von Klenze testifies Romberg's close ties to engineering circles at Munich. We may therefore safely assume that the details of the curved board roofs were familiar to the Munich architectural protagonists. Unfortunately, the 1833 edition of Romberg's book contained only minimal text, so that we do not know whether Romberg considered the curved plank roof as advantageous then or not.

Apart from Romberg's book, the single most influential publication on wood construction of that time was probably the series of „Vorlegeblätte für Zimmerleute", published under Beuth's direction at Berlin. While the first few editions (starting in 1827) were only intended for the use of the civil servants of the Prussian "Technische Deputation für Gewerbe" (the ministry of economic affairs), later editions were available to the general public after 1835 [12]. The "Vorlegeblätter" ("pattern designs") recommended curved plank roofs for all types of buildings requiring an unobstructed space. Detailed data on required scantlings were provided, as a function of span ([12] p. 9, pl. XII).

Critical attitudes towards the curved plank roof clearly emerge in the 1840ies. Carl August Menzel (1842) explicitly advised against any use of them. He also claimed that they were in fact only rarely executed in practice. However, for anybody who would insist on the application of the scheme, Menzel has special advice. Arched trusses are recommended only as subsidiary support for ordinary roof structures ([13] p. 136-137).

We do not know whether this change of opinion was brought about by a knowledge of Paul Joseph Ardant's careful scientific analysis of curved roof trusses of 1840 [14]. However, it is clear that Ardant's book found a quick reception not only in France, where it brought about the rapid end of the Émy arch, but also in Germany (a German translation appeared in 1847). Ardant had proven scientifically – both by an examination of experimental results obtained by Reibell in the late 1830ies and by careful application of Navier's theory of bending – that all the arched girders were much more flexible than trusses consisting of straight members. Even though Ardant did not explicitly speak against the arched trusses ([14], p. 9-13), it was now evident that they were both economically and structurally inferior to ordinary roof trusses such as Italian purlin roofs which had been used for centuries.

In the following decades, curved plank roofs continued to be carried out occasionally. The technical literature generally reflected Ardant's analysis; the older types of curved plank roofs were still reprinted again and again, but now this was generally accompanied by a note that these structures were mainly of historic interest. The widely read textbook of Breymann and Lang of 1870 [15] recommended arched girders only in the context of an overall purlin roof. As an example, Lang presented a roof over a gymnasium at Karlsruhe which he had executed himself (Fig. 10). Lang's roof has principal rafters and a horizontal iron tie to carry the thrust. The arch appears as a subsidiary reinforcement of the roof truss; it helps to support the purlins by means of radial struts. Longitudinal stiffening of the structure is achieved by wind braces at the principal rafters.

In his comprehensive textbook on construction, Rudolf Gottgetreu wrote in 1890 in retrospect about the development

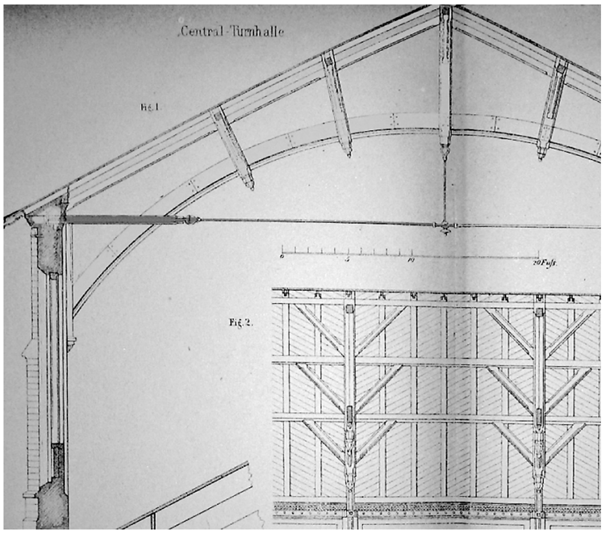

Fig. 10. Turnhalle Karlsruhe (1869). [15] Anhang Fig. 1

of the curved board roof: „Probably no other roof structure has been the object of more experimentation than the curved plank roof" ([16], p. 217).

3. SURVIVING EXAMPLES

3.1 Neuburg/Danube, Castle

The western aisle of the castle at Neuburg/Danube (Bavaria) is covered by a low-pitch roof that rests on curved plank trusses. The roof which spans 18,5 m has not received much attention until now, even though it is well preserved and one of the biggest surviving German curved board roof trusses (Fig. 11). The roof was erected in 1824 under the direction of Bernhard v. Morell, a former student of Weinbrenner at Karlsruhe, and built by the Neuburg court carpenter Wildenauer [17].

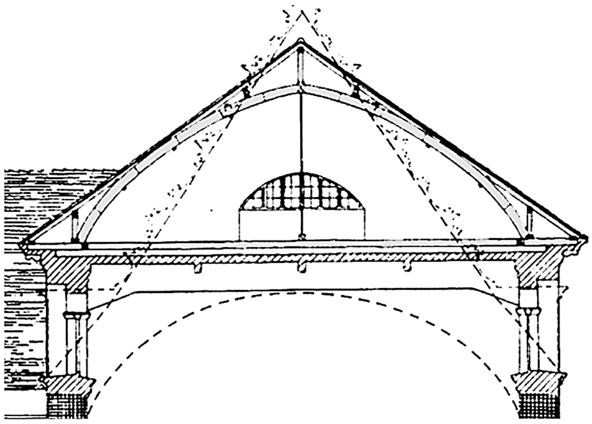

Fig. 11. Schloss Neuburg, Westflügel [17] S. 240

The roof has slightly pointed arches and a continuous ridge purlin. They consist of three layers of boards, connected by iron nails. No distinction between principal rafters and ordinary rafters has been made, all rafter pairs are identical. At the apex of the arch, the arched rafters are tenoned into a purlin, and the rafter feet are notched into a sill. The straight rafters which carry the roof cover are also continuous; they show a sawtooth-shaped step joint where they touch the arches, as in Fig. 2. The ridge of the roof is supported by a ridge purlin, which in turn rests – by means of intermediate struts – on the "ridge purlin" of the arches. The pair of ridge purlins – at the apex of the arch and straight rafters – constitute a kind of stiff "truss" parallel to the axis of the roof and provide longitudinal stiffening. Further longitudinal purlins support the straight rafters at their lower end; these purlins are carried by vertical struts. In the upper region, another intermediate purlin is provided, which is supported from the haunches of the arch. Further longitudinal stiffening is provided by pairs of strong boards clasping the arches, as shown in Fig. 4. These pairs are held together by bolts at irregular spacing. On second sight, it turns out that in fact the roof has continuous tiebeams only in every forth axis. A singular device are the iron ties by which the continuous tiebeams are suspended from the ridge truss.

The overall design clearly reflects the contemporary state of development: Traditional elements of Gilly's curved plank roof are present, but more recent improvements have also been adopted. The joint between the straight rafter and the curved rafter is reminiscent of Gilly's designs (Fig. 2). The pointed arch follows Gilly faithfully. On the other hand, more recent developments such as the paired boards clasping the arches are not only found in the Vienna roof (cf. Fig. 4), but also in the roughly contemporary wooden dome which Georg Moller erected above the catholic church at Darmstadt (1822-27, [18]). Morell and Moller may have met during their studies at Karlsruhe; maybe Morell got the inspiration for the curved plank roof at Neuburg there. Their common teacher Weinbrenner was explicitly against curved plank roofs, as early as 1809 ([1], p.110).

3.2 The harbour master's house and bard at Beilngries

In 1836, the Bavarian king Ludwig I. orderd to commence the construction of the canal betweeen the rivers Danube and Main. The canal was opened in 1843. The responsible engineer for the entire project was Heinrich Friedrich von Pechmann [19]. Interestingly, several curved board roofs were executed in the context of the canal. Here, we discuss the so-called harbour master's house and the adjacent barn at the former canal port (now dry) at Beilngries, Bavaria. Today, only the iron crane and the revetment of the former harbour basin testify of the former purpose of the buildings.

All the buildings in the architectural context of the canal were designed by Pechmann, but examined and revised architecturally by a commission at Munich headed by Leo von Klenze. Design drawings dated 1837 show the general plan of the harbour, as well as sections and elevations of the individual buildings (cf. Fig. 12 for a sample). The design drawings present conventional "stehender Stuhl" trusses and curved board roofs as alternatives. Fig. 12 shows a comparative example of an original drawing of a curved board roof in the harbour master's house in Bamberg.

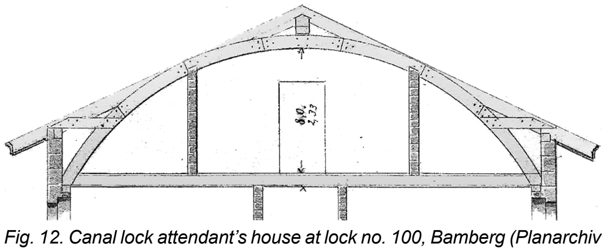

Fig. 12. Canal lock attendant's house at lock no. 100, Bamberg (Planarchiv Eisenbahnmuseum Nürnberg)

Both builings in Beilngries were also actually carried out with arched roof trusses, which have been preserved (Fig. 13 and 14).

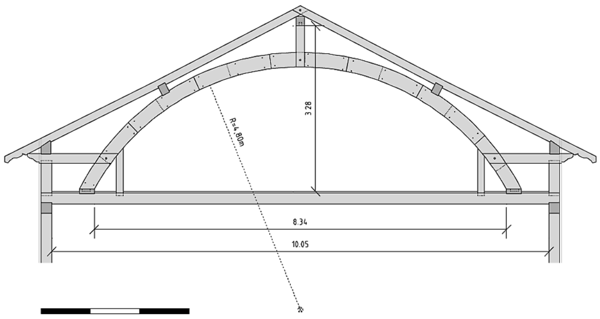

Fig. 13. Beilngries, barn

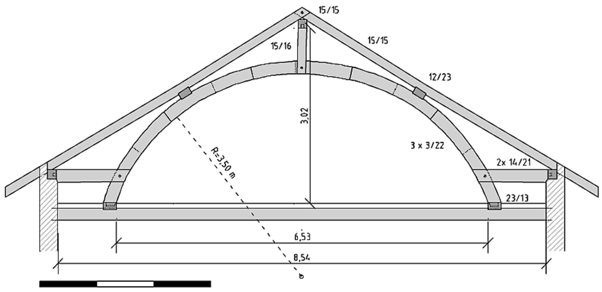

Fig. 14. Beilngries, harbour master's house

The ground floor of the harbour master's house (1837-43) contained the harbour master's living quarters. The attic is empty today, however, the design drawings show that a small chamber was planned here as well. Probably, this was a heatable chamber which contained a "economic stove" as invented by Pechmann. No traces of the attic chamber or of the stove are recognizable today.

The barn was erected in 1850 (archival note, Verkehrsmuseum Nürnberg). Both houses have a rectangular ground-plan and a ridge roof. The outer straght rafters are notched into a sill, supported by intermediate and ridge purlins, and halved across each other at the apex. The purlins are carried by curved plank arches, which rest on sills which are notched onto the tiebeams. The attic has raised eaves. The feet of the straight rafters are anchored at the arches by twin ties which clasp the arches and are attached by means of bolts. A pair of boards also clasps the apex of the arch and attaches it to the ridge purlin.

Both roofs thus have continuous tiebeams. The longitudinal stiffening of the roofs is achieved by the purlins; only in the barn, we find additional wind braces. The arches are part of principal trusses; every third pair of rafters is a principal truss. The board arches are assembled by means of iron nails. At the barn, the nails are clearly visible. At each side of joint, there are two nails. An additional pair of nails is in the middle of each board.

The barn has a span of 8,54 m and is therefore somewhat larger than the harbour master's house (span 6,53 m). This difference in span is probably the reason why the arches of the barn consist of three layers of boards, whereas those of the harbour master's house have only two layers. The arches are semi-circular. The length of the boards is 1,50 m, their height 28 cm at the barn. At the harbour master's house, the boards are 1,60-1,70 m long and 22 cm high.

The curved board roof system was quite probably selected because an unobstructed roof space was desired. Both buildings require economic storage space. In constructive detail, the roofs at Beilngries are clearly different from Gilly's original layouts, and they reflect the state of the development around 1840 well. Johann Andreas Romberg published similar roofs in the 1833 edition of his textbook ([7] pl. LXXII).

3.3 Brine reservoir at Klaushäusl (Grassau)

The brine reservoir at Grassau-Klaushäusl is part of the brine pipeline from Reichenhall to Rosenheim (constructed in the second decade of the 19th century). It is part of one of the seven pumping stations of that pipeline. The brine arrived at a lower container and was pumped up to a tank high on the hill. This tank was covered by the building which we discuss here (for a historyof the pipeline and its technical equipment, see [20-22]).

The present building (Fig. 15) dates back to a reconstruction of 1875 [23], and its roof is therefore an interesting late example of a board arch roof. Already Gilly himself had suggested the use of his timber arch roofs for building like brine reservoirs, pumping stations or steam engine shelters (cf. [1], p. 34).

The one-storey building has a rectangular plan, ashlar masonry walls, and is covered by a ridge roof. The width of the single room in the interior is 6,20 m. The continuous rafters are notched into sills at their base, supported by two intermediate purlins, and halved across each other at the top. The purlins rest on semicircular board arches. The arches are tenoned into a sill. The eaves are again raised, resulting in a low-pitch aspect from the outside. Between the sill and the wall-plate, a crippled tie-beam is clamped, which joins the arch to the rafter feet. The arch is attached to this brace by a bolt.

The lack of a tiebeam is balanced by the thickness of the walls. They are 72 cm thick. Longitudinal stiffening of the construction is provided exclusively by the purlins. There are 7 arches, and there are 2 rafter pairs in each bay between the arches, so that the arches can be called principal trusses. The purlins rest on massive gable triangles.

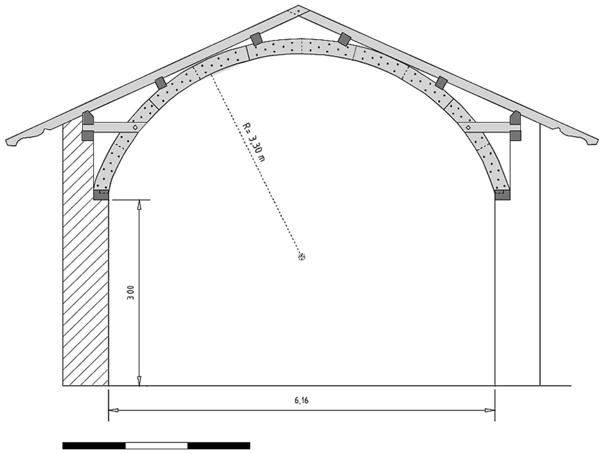

Fig. 15. Grassau (Bavaria), Klaushäusl, brine reservoir

The construction of the arches follows Gilly's suggestions precisely: At the joints, we find iron nails, whereas each board is fixed to the layer beneath it by a treenail in its center as well. The arches consist of three layers of boards. Each layer is 4 cm thick. The boards are approximately 1,20 m long and 25 cm high. Compared to the scantlings suggested by Friderici in 1811 in a detailed list ([4], p. 129), we find the dimensions as built are considerably stronger.

The roof contains all the „modern" elements which we have encountered in the review of the 19th century literature on arched timber frames. Arches and rafters are separated, the overall roof layout is a purlin roof. The construction is not dramatically different from the harbour master's house built 40 years earlier. This testifies to the decreasing attention which the scheme received during the second half of the 19th century. Compared to Lang's gymnasium roof of 1869 ([15], cf. Fig. 10), the almost contemporary roof at Klaushäusl looks a bit old-fashioned.

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Abstract

Curved roof trusses made up by boards were first introduced by Philibert de l'Orme in 1561. However, de l'Orme's invention did not succeed much until it became popular in the late 18th century. David Gilly, a civil servant in the Prussian administration of buildings, promoted the idea by a series of propagandistic publications. In the years around 1800, a considerable number of curved plank roofs were actually built, ranging up to spans of around 20 m. While the history of these roofs is fairly well known, the later development of the curved plank roof is less known. The system was severely criticized by early engineer such as Paul Joseph Ardant in the 1840ies, and scientific arguments were put forward against it. Nevertheless, curved plank roofs continued to be used until the late 19th century. There was even some Renaissance of the scheme in the last third of the century, probably mainly due to curved plank roofs published in well-known construction manuals such as Gustav Adolf Breymann's.

We present different curved plank roofs which are still preserved in Bavaria. They cover the time range between 1824 (Neuburg / Danube, castle) and the 1870ies (brine reservoir building on the Reichenhall-Rosenheim saltworks pipeline). A considerable number of curved plank roofs has also been preserved along the canal between the Danube and the Main (1840ies), in the context of lock attendants' homes. The structures will be presented, compared to earlier curved plank roofs, and put into the context of contemporary technical literature.

Streszczenie

Krążynowe kratownice zbudowane z drewnianych elementów po raz pierwszy wprowadził Philibert de l'Orme w 1561 roku. Jednakże wynalazek de l'Orme'a został spopularyzowany dopiero pod koniec XVIII wieku. David Gilly, urzędnik państwowy w pruskiej administracji budowlanej, wypromował ten typ konstrukcji w serii publikacji. Około roku 1800 istniała już znaczna ilość dachów krążynowych, a rozpiętość niektórych z nich wynosiła nawet około 20 m. Podczas gdy historia tych konstrukcji jest dość dobrze znana, to ich dalszy rozwój już znacznie mniej. System ten był krytykowany przez ówczesnych inżynierów, takich jak Paul Joseph Ardant, w latach 40-tych XIX wieku i wysuwano przeciwko niemu argumenty naukowe. Jednakże dach krążynowy był nadal stosowany, aż do końca XIX wieku. Nastąpił nawet pewien renesans tego modelu konstrukcji w ostatnim trzydziestoleciu XIX wieku, spowodowany zapewne publikacjami na temat dachów krążynowych pojawiającymi się w dobrze znanych podręcznikach konstrukcyjnych, takich jak dzieło Gustava Adolfa Breymann'a.

My przedstawiamy inne dachy krążynowe, jakie są nadal zachowane w Bawarii. Pochodzą one z okresu pomiędzy rokiem 1824 (zamek Neuburg nad Dunajem) i latami 70-tymi XIX wieku (budowa zbiornika solanki przy rurociągu warzelni soli w Reichenhall-Rosenheim). Znacząca liczba dachów krążynowych została także zachowana wzdłuż kanału łączącego rzeki Dunaj i Men (lata 40-te XIX w.), w domach dla operatorów śluz. Konstrukcje te zostaną zaprezentowane, porównane z wcześniejszymi dachami krążynowymi i skonfrontowane z współczesną literaturą techniczną.

Anu Soikkeli[1], Jussi Tervaoja[2]

World's largest wooden church in Kerimäki

Największy drewniany kościół świata w Kerimäki

Keywords: Heritage, Timber, Log, Church, Structures

Słowa kluczowe: dziedzictwo, drewno, bal, kościół, konstrukcje

1. CONSTRUCTION OF THE CHURCH

Construction of a new church was planned in Kerimäki since the 1810s, as the old church had become isolated. According to an ordinance given in 1823, the church was to be built of stone. Construction was postponed because the parishioners understood that building a stone church would be very costly. In the end the emperor granted permission to use timber as the building material [1].

According to the first church plan, only 1500 parishioners would have fit in the church. The residents of Kerimäki did not approve the plan and they appealed to the intendant's office for plans for a larger church. Initially the office did not consent to this request if the building material were timber. It was feared that a very large wooden church would not hold together [2]. It was also doubted that the parson's voice would carry enough in a large church. Finally, in 1844 architect A. F. Granstedt designed a church that would accommodate 5000 people, and construction began immediately [3].

The parishioners committed themselves to a labour-intensive, costly undertaking. The construction work was supervised by distinguished church builder Axel Magnus Tolpo until his sudden death, after which his 23-year-old son, Theodor Tolpo, continued his father's work. The parishioners also took part in the building process according to their income bracket – every man between the ages of 15 and 60 was obliged to participate in the building. The parish appointed professional builders, masons, blacksmiths and carpenters. [4] The parishioners paid these professionals' wages in grain and money.

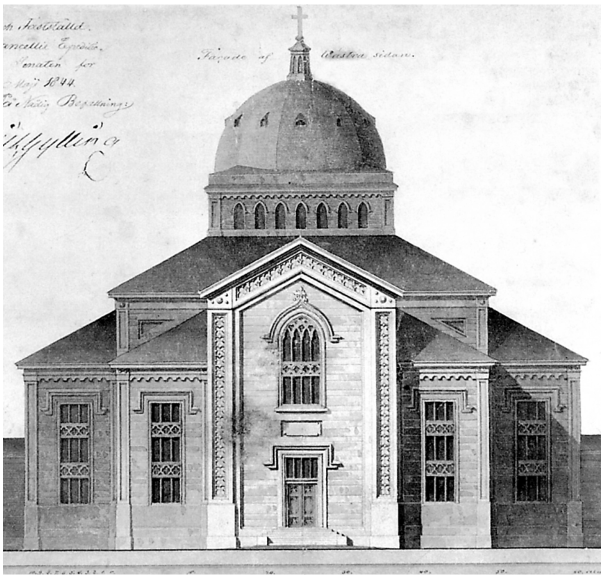

Fig. 1-2. The original drawing of the church by architect A. F. Granstedt. Kerimäki church at the beginning of 20th century (Archive of Kerimäki parish)

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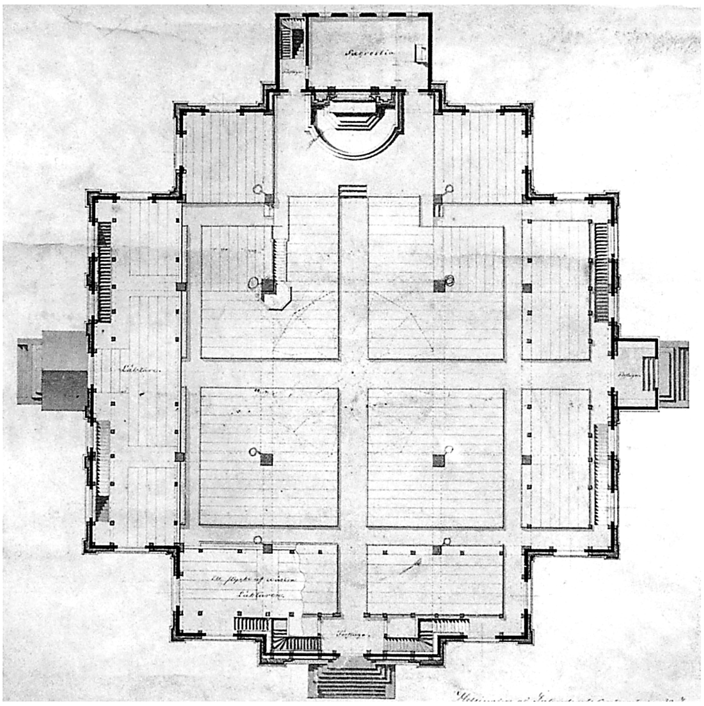

Fig. 3-4. The interior of Kerimäki church (Photo Anu Soikkeli, 2011). The floor plan of the church is a short-armed double cruciform church (Archive of Kerimäki parish)

Construction began in the spring of 1845. Selected stones and timber material from around the parish were brought to the building site during the previous winter. Logs were transported along winter roads by horses or floated along waterways in the summer. [5] Boards were primarily sawed by hand at the building site, as the parishioners didn't want to buy boards from sawmills owned by people from other parishes. The parishioners fulfilled their work obligation without pay, but the craftsmen were paid in grain. Granstedt estimated that 294,000 nails were forged at the building site [6].

Even while the church was being built there was considerable doubt about its structures, but in the end permission was given to install windows in the centre dome. The roof was covered for the most part with wood shingles, since the parish had only enough money to cover the dome with sheet metal. The church was completed in the autumn of 1847 and was consecrated the following summer, when it was also painted [7]. The church was completed relatively quickly, as there was plenty of manpower and workdays were long – 14 hours in the summer and 10 in the winter [8]. A tall bell tower was built at the same time near the main entrance of the church. Its lower part is made of stone and the upper part is made of wood.

Attempts have been made to explain the large size of the church as a result of confusion between ells and metres or a mix-up of drawings. In actuality the size of the church was dictated by the size of the parish: according to the rector back then, the church had to accommodate half of the 12,000 parishioners at a time.

2. ARCHITECTURE AND STYLE OF THE CHURCH

The mid-1800s was a period of change in European church construction. Styles and details in church designs were taken from classical, Gothic, Romanesque and Byzantine styles. Granstedt's own designs were influenced by the cathedrals in

Fig. 5-6. The exterior and interior are extremely well preserved (Photo Anu Soikkeli, 2011)

Firenze and Aachen, for example, as well as Hagia Sophia [9], but the old Finnish church building tradition is also recognizable in the church.

The Kerimäki church is huge: it is 45 metres long, 42 metres wide and 27 metres high at the centre dome. The tip of the cross atop the dome is 37 metres high. The roofs of the arms of the transept are higher than the roof of the twin galleries built as a continuation of the arms. There are seats for 2870 parishioners in the nave, which is divided into three aisles by four massive pillars, and under the galleries. An additional 1830 parishioners fit in the two-storey galleries located in three arms of the cruciform structure; altogether there are seats for nearly 5000 people [10].

The floor plan of the church is a short-armed double cruciform church. The dome is supported by four timber pillars at the centre of the cruciform shape. The pillars are reinforced by richly profiled tie-beams connected to the walls. Open roof trusses shaped like three-leaf clovers are also partly supported by the tie-beams. The nave is well-lit, as three-storey windows in the walls and openings in the dome give abundant light, adding to the feeling of spaciousness in the nave. The church is flooded with light from the three-storey, diamond-patterned vertical windows, upper windows in the east and west gables and side-by-side lanceted windows in the centre dome. The pulpit was situated on the left side of the main aisle of the church on one of the large pillars at the centre of the cruciform shape. The parish clerk led the singing of hymns from a chair next to the altar. The parish clerk's chair was no longer needed after 1894, when a 20-stop pipe organ was installed in the west gallery of the church.

The horizontal log walls of the church are clad with horizontal boarding. The influence of Romanesque and Gothic styles are apparent in the moulding and decorative woodwork of the facades. The corner pilasters imitate stone churches, the friezes carved with a quatrefoil motif framing the main entrance and the arched windows in the east and west gables mimic stonework. The interior decoration shows traces of Gothic features.

3. LATER MODIFICATIONS

The original untreated wood-shingle roof did not last long, and it was covered with galvanised sheet metal in 1904. The green shade of the dome mimics a copper roof; elsewhere the roof is painted black. The footing was repaired with cement mortar and the steps of the side entrances were replaced with concrete steps [11]. The surroundings of the church, which was built on an open hill, have gradually become overgrown with trees, and the church is partly hidden by them.

In 1932 the church was expanded with a smaller nave; a so-called winter church with seats for 200 was built on the side of the church. This winter church was further expanded in 1953, after which it seated 250 [12]. The main nave of the large church did not have heating for the cold season, and large iron stoves installed in 1915 were not able to heat the church, either [13]. Every year on Christmas morning a Christmas service is held in the big church, where according to an old tradition everyone wears a fur coat and mittens during the service. The church has several decorative iron stoves, but they have been unused for over 50 years, already.

The exterior of the church was repainted in the summer of 2009. The surface area of the painted walls was about 4000 m^2. The painting company estimated that 4500 litres of linseed oil paint was used for the job [14]. The previous primer paint was zinc white, which was coated with oil paint. The zinc paint had since hardened and become chalky, which along with changes in the weather made the impermeable layer of paint flake off. Raising funds for the paint job was said to be challenging, as the cost rose to €813,000. The church has to be painted again in about 20 years.

4. STRUCTURE

4. 1. General characteristics and limitations of a log structure

Traditional Finnish log construction was based on a saddle-notch corner joint system, which spread to Finland from the Russo-Byzantine culture in the east during the Iron Age [15]. The saddle-notch corner joint structure is made by notching the top of one horizontal log and the bottom of another, forming an interlocking corner when placed one upon the other. These joints in the crossed logs lock the courses of logs into a sturdy framework, and for this reason a log building should have a sufficient number of interlocking joints. To ensure that the logs are tightly seated against each other, a longitudinal groove is cut along the bottom of the upper log. This groove also prevents rainwater from entering the seam between the logs. The logs are fastened to each other by means of wooden dowels inserted into vertical holes drilled through adjoining logs.

Fig. 7-8. Short corner and long corner of old Finnish log buildings

Of Finnish wood species, pine, spruce and aspen were suitable as logs, although pine was usually preferred because it had the best properties for this type of construction [16]. According to traditional knowledge, trees used for logs were felled in winter, when it was easy to transport them in the forest and the barked logs had time to dry sufficiently during early spring before fungi had a chance to begin growing in them. Timber was readily available, which was one reason why logs were long the most common building material. The oldest preserved wooden churches date back to the 1600s.

Settling of the walls during the shrinking period of the logs is a peculiarity of log construction. Suitable structural designs which take this phenomenon into consideration have been developed over the centuries. This allowance for the shrinking period also affects the building's architecture [17]. The shrinking period of a massive log wall is 3-4 years, and this period cannot be artificially shortened. The wall settles due to shrinking of the wood and tightening of the seams and joints; the logs never fit together as tightly as possible during the construction phase. Tightness of fit can be improved during the construction phase by using dowels, for example. Tight-fitting dowels can be used to draw the logs against each other and prevent the seams from opening when working on the next courses. Most cracks left during the construction phase will close during the first winter as a consequence of the load of snow. The logs dry out during the shrinking period, resulting in the appearance of small cracks, which cannot and need not be prevented. Careless workmanship or poor-quality wood may cause the seams to open [18]. Thus, in the centuries-old log construction tradition, logs have always been chosen carefully and the grooves cut into the logs have been stuffed with moss and later with flax stuffing.

In the saddle-notch corner joint system the size of the building is dictated by the length of the logs. Logs longer than 10 metres have been used in Finland only in exceptional cases. The appearance of double cruciform churches in the late 1700s was brought about by the length of logs; the objective was to build as large a nave as possible using 5–7.5-metre logs. The length of the logs was dictated by both tree growth and the weight of the timber material, which had to be taken into consideration when building with manpower. It should be noted that full-length logs were only used above and below window and door openings [19].

Fig. 10-11. The pillars are reinforced by richly profiled tie-beams connected to the walls (Photo Anu Soikkeli, 2011)

4.2. Structural system of the Kerimäki church

The floor plan of the church is a short-armed double cruciform church (Fig. 4). The dome is supported by four timber pillars at the centre of the cruciform shape. The pillars are reinforced by richly profiled tie-beams connected to the walls. Open roof trusses shaped like three-leaf clovers are also partly supported by the tie-beams. The nave is well-lit, as three-storey windows in the walls and openings in the dome give abundant light, adding to the feeling of spaciousness in the nave.

The outer walls of the church – which support the roof structure – are made of horizontal logs and the interior pillars are each made of four vertical 15-metre-long, 16-inch-thick (40 cm) posts which are bolted together and made rigid in the horizontal direction by the side galleries. This design was structurally challenging, as the difference in settling between the vertical and horizontal structures in the outer walls and the pillars was over 30 cm. This difference in settling was not resolved with adjustable joints as is done nowadays; instead the difference has been absorbed by the large scale of the building or it was taken into consideration in advance.

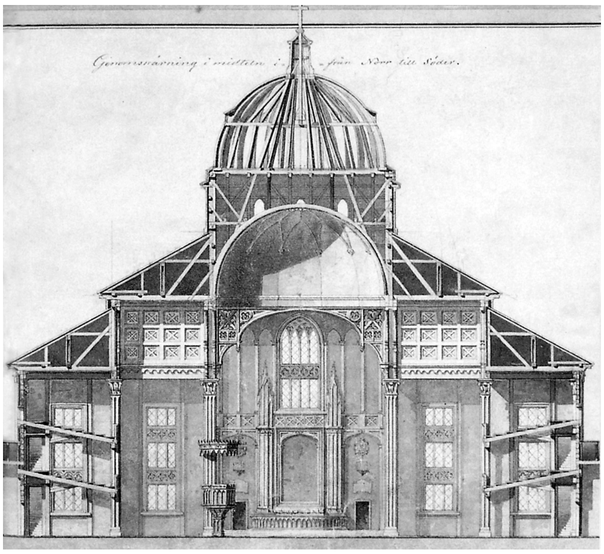

Fig. 9. The walls, punctuated by tall windows, are made rigid with decorative vertical wooden supports hidden in the walls and which allow for settling of the log structure. The log walls form panels which give the building torsional rigidity (Archive of Kerimäki parish)

The designer of the church sought to keep the cohesive log structure of the whole church as rigid as possible. The height of the level log framework is about 15 metres. The walls, punctuated by tall windows, are made rigid with decorative vertical wooden supports hidden in the walls and which allow for settling of the log structure. The log walls form panels which give the building torsional rigidity. Beams carved with a quatrefoil motif frame the arched windows, adding rigidity to the frame.

The roof is comprised of thirteen structurally independent components. Above the level log frame the roof surfaces rise in two steps. The highest, central component of the roof structure is formed by the wooden dome.

Italian architect Andrea Palladio (1508-1580) had developed various truss and girder structures already during the Renaissance period. The roof structures of this building also employ truss analogy, although not fully, as the roof structures also contain curved structural components and stabilising tie-beams. The joints of the beams are so-called contact joints, made with the carpentry skills of that time: dovetail or notched joints locked with wooden pegs and tightened, if necessary, with forged bolts.

5. CONCLUSION

The Kerimäki church is a monument in which the constructive potential of timber material – in view of the circumstances of the time – was exploited to the extreme. When it was built, the ingenious double cruciform structure of the church was 80 years ahead of its time and in all a masterpiece of carpentry skills of that time. Church services are still held in the church during the summer, and it is also a venue for concerts and a popular tourist attraction.

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Abstract

The Kerimäki church is the world's largest wooden Christian church and it has the most spacious interior in Finland. The church was designed by architect A. F. Granstedt; its construction took three years and was completed in 1847. The parishioners had to take part in the building process according to their income bracket. Every man between the ages of 15 and 60 was obliged to participate in the building.

The Kerimäki church is 45 metres long, 42 metres wide and 27 metres high. The height of the dome is 37 metres and there are altogether 1670 metres of pews. The seating capacity is for over 3000 people, but standing room included, the church can accommodate 5000 people. According to the floor plan it is a short-armed double cruciform church. The church is a miracle of its own time, a masterpiece of carpentry with its pews, columns, galleries, tie-beams, arches, domes and lanterns.

The church was originally intended to be built of stone, but because of its high cost the parish was allowed to build the church using timber. However, the exterior of the church imitates a stone building in many ways. The log walls of the church were clad with horizontal boards. The corner pilasters imitate stone churches, and the friezes carved with a quatrefoil motif framing the main entrance and the arched windows in the east and west gables mimic stonework.

The emphasis of this paper is on the unique construction system and architecture of the church.

Streszczenie

Kościół w Kerimäki jest największym drewnianym kościołem chrześcijańskim na świecie i ma najbardziej przestronne wnętrze w Finlandii. Kościół został zaprojektowany przez architekta A. F. Granstedta, a jego budowa zajęła trzy lata i została ukończona w 1847 roku. Udział parafian w procesie budowania kościoła był uzależniony od ich statusu majątkowego. Każdy mężczyzna w wieku od 15 do 60 lat był zobowiązany do pomocy przy budowie.

Kościół Kerimäki ma 45 metrów długości, 42 metry szerokości i 27 metrów wysokości. Wysokość kopuły wynosi 37 metrów, a całkowita długość ławek to 1670 metrów. Przewidziano 3000 miejsc siedzących, a jeśli weźmie się pod uwagę również miejsca stojące to kościół może pomieścić 5000 osób. Budynek został zaprojektowany na planie krótko-ramiennego podwójnego krzyża. Kościół był cudem swojej epoki, ciesielskim arcydziełem z ozdobnymi ławkami, kolumnami, galeriami, belkami stropowymi, łukami, kopułami i latarniami.

W pierwotnym zamierzeniu kościół miał być zbudowany z kamienia, ale z powodu wysokich kosztów materiału parafii pozwolono zbudować go z drewna. Niemniej, od zewnątrz kościół na różne sposoby imituje budynek kamienny. Jego ściany z bali zostały pokryte poziomymi deskami. Narożne pilastry imitują te stosowane w kościołach kamiennych, podobnie jak motyw czteroliścia wyrzeźbiony na fryzach, otaczających główne wejście i łukowe okna na wschodniej i zachodniej ścianie szczytowej.

Szczególny nacisk w tej prezentacji został położony na unikatowy system konstrukcyjny i architekturę kościoła.

David Wendland[1]

Research on "cell vaults": analytic and experimental studies on the technology of late-gothic vault construction

Badania „sklepień komórkowych": analityczne i eksperymentalne badania konstrukcji późnogotyckich sklepień

Keywords: Vault Construction, Historical Construction Technique, Survey, Experimental Archaeology, Shape Modelling, Teaching Historical Construction Technique

Słowa kluczowe: konstrukcja sklepień, historyczna technika konstrukcyjna, ekspertyza, archeologia eksperymentalna, modelowanie kształtu, uczenie historycznej techniki konstrukcyjnej

1. INTRODUCTION

In the decades around 1500, builders and patrons all over Europe felt a particularly fascination for the design and construction of vaulted ceilings, searching for ever more challenging, daring, complex and original solutions. In England, the complex rib vaults are developed to fan vaults and further to pendant fan vaults, such as in the Henry VII Chapel in Westminster Abbey. On the continent, the vaults with hanging keystones and "air ribs" first introduced by Peter Parler in St. Vitus Cathedral in Prague became popular. Rib vaults appear with ribs describing double curvature like looping ribbons. These are just some of the many inventions in vaulting of that period.

In this atmosphere, around 1470 the "Cell Vaults" according to common belief were invented first for the construction of the Albrechtsburg, the new palace of the princes of Saxony at Meissen (Germany) [1-3]. These vaults that sometimes have stone ribs but in many cases only sharp groins, are characterized by their folded surface with ridges between the groins (Figure 1), creating ceilings with subtle patterns of light and shadow. The construction material (apart from few exceptions) is brick masonry. Their design basically corresponds to that of late Gothic net vaults, but the pattern of their groins and arches is often much more complex than that in conventional rib vaults. "Cell Vaults" rapidly became popular first in Saxony and then spread all over central-eastern Europe, including Bohemia, Poland, the Baltic, Prussia, and even beyond. During more or less a century hundreds of them were built in palaces, town houses, convents and churches.

From sources and contemporary records, we learn nothing about how these vaults were built, just as little why they were introduced and why they became so popular in such a vast area. We don't even know how these vaults were called – the names we use are of modern origin. Hence, on the motifs we can only guess, but we tend to suppose that they were manifold. Regarding aesthetic motifs, the folded surfaces show some relationship with contemporary sculpture. Further, this type of vaults certainly has great benefits in the task of designing rather low, wide spanned vaulted ceilings, like in the Albrechtsburg. Technological reasons

Fig. 1. Cell vault in the second floor of the Albrechtsburg at Meissen (Germany). The folded surface of such vaults are built in brick masonry (D. Wendland)

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could regard the acoustic qualities and the assumed structural performance. But most scholars since the 19th century believe that the main motif is related to the construction process, which is supposed to be greatly rationalized in this type of vaults, compared to the conventional rib vaults with domed webbings.

This hypothesis, first formulated by G.G. Ungewitter, is based on the assumption that the "Cell Vaults" due to their particular shape could be built free-handed, i.e. without the support of a formwork under the whole surface during construction, in a very straight-forward way, supporting only the groins by centering arches [4]. Ungewitter also believed that by this innovation, the need of cutting bricks ad hoc to the right shape would have been at least greatly reduced.

However, as pointed out, in the sources there is no information whatsoever about the building process and the design criteria of "cell vaults" – hence no contemporary records can be found that could support this hypothesis. Therefore, it is necessary to search other ways in order to find out how five centuries ago the "cell vaults" were, or might have been, actually built.

2. CONSTRUCTION PRINCIPLES OF LATE GOTHIC "CELL VAULTS"

2.1. Problematic of the current technical description

To understand the building process is also necessary for explaining the peculiar appearance of the "Cell Vaults", in particular the curves of its groins and ridges, and the shape of the vault surface. And it certainly is of central interest if we want to gain understanding on the design, and on the communication process which took place between the patrons, architects and craftsmen. Hence, this technical issue gains importance also for the discussion of these artifacts from the point of view of Art History.

As a matter of fact, all technical descriptions provided in the scientific literature can be drawn back to Ungewitter's hypothesis, hence, have no documental base in the period when the building of "Cell Vaults" was practiced [4]. Moreover, critical revision shows that it is also poorly based on archaeological evidence. Most of the graphical representations in the literature are idealized, especially regarding the curves of the ridges, replacing the actual state with the ideas molded by the reception of the description model founded by Ungewitter. Detailed surveys describing the geometry and the details of these vaults with fidelity are lacking. And even the modern attempts to reproduce these vaults, instead of the archaeological evidence are based on the modern description model – a situation with an enormous risk of circular reasoning.

2.2. The new research approach: Experimental archaeology correlated with detailed surveys and analyses of the archaeological evidence

However, in order to formulate hypotheses on the construction process that is not otherwise documented, its reproduction according to the method of experimental archaeology is the most promising. In the current research on the construction of "cell vaults", experiments were carried out that were closely linked with detailed surveys of original

Fig. 2. "Cell Vault" in Trebsen (Germany), where the curves, surfaces and also the masonry textures could be surveyed and analyzed (D. Wendland)

Fig. 3. Detail of the analyzed vault, showing the geometric complexity of the curve described by the inward fold, and the continuity of the masonry texture; both in contrast to the current description model (D. Wendland)

vaults – in particular one vault where in large portions the masonry texture was visible (Figure 2, 3). The curves, surfaces and single courses in the masonry texture were subject of 3-D-surveys and geometric analyses that were carried out by means of a software tool for Reverse Geometric Engineering. By these analyses the construction process could be characterized, principle statements on the temporary auxiliary structures needed for the building could be made, and construction principles based on the archaeological evidence formulated. The methodology of such analyses has been developed in earlier research by the author [5, 6].

By putting these hypotheses in practice in the experiments carried out in full scale, they could be evaluated and refined, and on the other hand, the practical experiences also lead to further observations on the object.

2.3. Analyzing the construction principles of "Cell Vaults"

From the geometric features of the curves, surface form and masonry texture, it is possible to learn about the building process and the principle features of the centering system. First of all, the geometry of the groins shows that during construction these must have been supported by single centering arches, not by a continuous formwork. Their curves can be described in simple geometric terms, in contrast to the shape

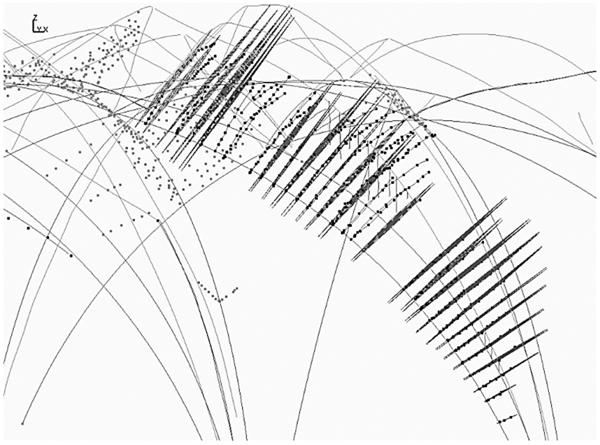

Fig. 4. Geometrical analysis of the masonry texture in the vault: the single courses are inscribed in tilted planes that over great portions are parallel, as usual in free-handed vaulting (D. Wendland)

of the vault surfaces. All arches describe circle segments in vertical planes with discontinuities at the intersection points, which also gives some indication on the whole system of centering arches. Such layout of the arches is common also in the contemporary rib vaults, and it corresponds to what we know about the geometric design of late Gothic vaults [4, 5, 7-8].

The ridges or inward folded groins, in contrast, describe curves with a complex geometry: the three-dimensional curves show variations in the direction and radius of curvature, and also abrupt changes of direction. From this we can conclude that they were not guided by centering, but resulted spontaneously in the process of the construction of the vault surfaces, which therefore must have taken place free-handed.

The analysis of the masonry texture proves that the analyzed vault was actually built free-handed. This is on one hand clear from the spatial position of the masonry courses, which, over great portions, are inscribed in tilted planes which are parallel (Figure 4) – a typical feature of the masonry texture of vaults built without formwork [5, 6]. Of particular interest are some corrections in the direction of the courses that can be observed (Figure 5): they were carried out where curvature had occurred in the beds that compromised the stability of the fresh course. By inserting bricks cut ad hoc in the shape of wedges, once more tilted plane courses were achieved that allowed to continue the building of the vault surfaces without support also above that point.

While all this confirms Ungewitter's basic hypothesis on the free-handed execution of "Cell Vaults", it is important to point out the fundamental differences that emerge between the analyzed vault and the description model formulated by Ungewitter and generally adopted in the following [4]. First of all, the masonry texture in principle consists of parallel instead of radial courses. Then, the vault surfaces don't correspond to the geometrically defined surfaces supposed by Ungewitter, but must be described as free form surfaces. Consequently, the ridges or inward folded groins differ from the smooth intersection curves of regular surfaces (Figure 6). And finally, the masonry texture in these folds is fundamentally different from Ungewitter's description, as, in spite of what in modern terms we would claim to be a regular bond pattern, usually the courses run continuously across them (Figures 3, 8).

Therefore, the rules for the masonry texture used by the builders of the "Cell Vaults" (and in late Gothic vault masonry in general) must be revised. As fundamental motif we must consider the continuity of the surfaces flow from the springing to the summit and the continuity in the masonry texture. The practical side of this could be clarified in the experiments.

2.4. Experimental reconstruction of late Gothic "Cell Vaults"

The experiments were carried out in collaboration with an academy of historical craftsmanship, reproducing two vaults in full scale with bricks and mortar according to the original. The used bricks have the same large format as the original, and are produced in the traditional manner – their porosity is of benefit for free-handed vaulting, and they can be easily cut to shape with an axe. The mortar has been reproduced according to the analysis of the original mortar.

In these experiments, the hypotheses on the construction principles as developed from the surveys turned out to be practicable, confirming the observations made above. The geometric problems in the masonry texture due to the curvature of the vault surface, have occurred in the experiments in the same manner as observed in the analyzed vault. Modern masons due to their training tend to turn the bed joint planes to a radial

Fig. 5. Corrections in the direction of the courses carried out ad hoc by cutting bricks, indicate that the vault was constructed without formwork D. Wendland)

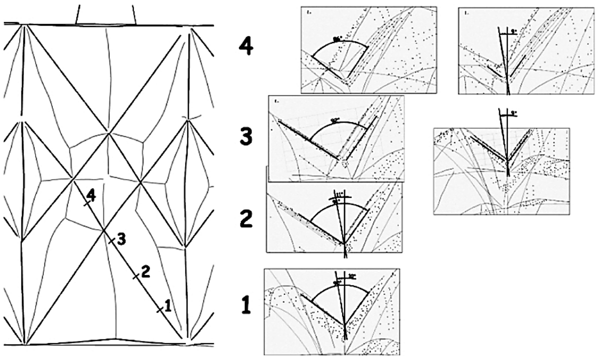

Fig. 6. Left, plan of the vault, showing the groins (black) and the negative edges (grey) in their geometric complexity (survey). Right, analysis showing the angle in the groin normal to the course and of the angle between the bisector and the plane of the groin. While in the cases shown here, the groins are approximately perpendicular, the angle to the plane of the groin arch systematically differs from 45°, contradicting the current description model (D. Wendland)

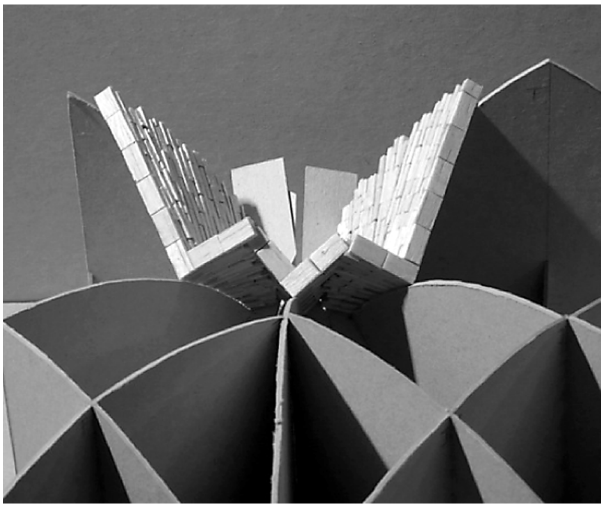

Fig. 7. Model simulation, showing that the angle of the courses at the groin (V-shaped connection in front) is not determined a priori, but depends on the development of the curves of the formeret arches (in the back) (D. Wendland)

Fig. 8. The exposed masonry texture of the analyzed vaults shows the continuity of the courses over large portions, across the discontinuities of the vault surfaces (cf. also Figure 3). This is not conform with modern masonry rules and contradicts the current descriptions models, but in the experiments turned out to be very practical (D. Wendland)

Fig. 9. Due to the continuity of the masonry texture, the courses are not arranged perpendicular to all groins, as usually assumed. Such observations lead us to understand the construction principles and rules applied by late Gothic masons (D. Wendland)

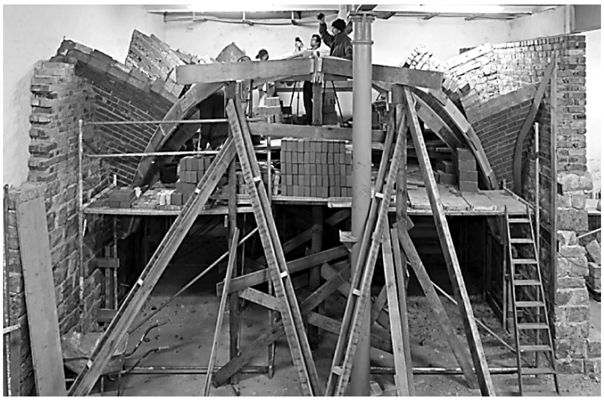

Fig. 10. Experimental reconstruction of the analyzed late Gothic "cell vault" (Figure 2), where the hypotheses on the construction principles and building process could be verified and refined (A. Gosch)

Fig. 11, 12. Details of the experimental reconstruction. The vault masonry is executed without formwork on single centering arches supporting the groins. The thickness of the shell is according to the archaeological evidence. The peculiar masonry texture with the continuity of the courses over the inward folds, as observed on the original vault, turned out to be practical (D. Wendland)

position by augmenting the mortar joints in the portions more distant from the groin. This attempt, however, turned out to be insufficient, apart from not occurring in the original where the thickness of the mortar joint is constant. Consequently, the same "hanging" courses resulted as in the analyzed masonry texture, requiring corrections ad hoc in the direction of the courses.

Also the continuity of the masonry texture over large portions, which we could observed and which we believe to be a principle of late Gothic vault masonry, was even surprisingly easy to reproduce in practice, just by slightly bending the beds and in sometimes cutting the corners of the bricks. The axe to cut bricks is needed very often in the process (once more contradicting Ungewitter's theory), but the traditional bricks are easily cut as they are rather soft and not brittle.

The main difficulty for everybody was to put apart the modern rules of bond pattern. These rules, although considered "historical" or "traditional", have been formulated only in the course of the 19th century and are not relevant for earlier historical periods. As already pointed out, from the archaeological evidence the construction principles which were used by the masons of the late Gothic vaults could be characterized, and in the experiments could be demonstrated to be very practical in execution. In reality, these principles also make a lot of sense, as with the continuous texture a very good coherence throughout the masonry is obtained. This can be illustrated by the fact that "cell vaults" built in the 19th century with modern criteria often show cracks that almost never occur in the original late Gothic vaults. In conclusion, we can now describe the difference between late Gothic and modern construction rules, and we are capable to reproduce late Gothic vault masonry with high fidelity.

2.5. Some conclusions regarding the building organization and the general design principles in late Gothic vaulting

Finally, the practical experience also gives way to statements regarding the building process on the whole and its organization, which are relevant not only for the construction of "Cell Vaults", but for late Gothic vault construction in general.

First of all, this regards the centering device. Except from very few fragments, no centering of late Gothic rib vaults or

"Cell Vaults" is preserved. However, a principle characterization of the temporary support construction that served also for the geometric control of these complex structures could be obtained by correlating the geometric analysis of the surveyed vault (and also survey data from rib vaults), with a source describing the construction of rib vaults in the 16th century [8] and information contained in the building accounts [9]. These considerations were also evaluated and further developed in the experiments, leading to a hypothesis about how the centering devices of late Gothic vaults must have looked like (Figure 15).

Another consideration relevant for late Gothic vault constructions in general regards the organization of the building process. As a matter of fact, it would not have been possible to build the analyzed "Cell Vault" by construction one bay after the other because, as pointed out, the masonry texture is continuous over the whole squinch. Therefore, starting from the springer, single portions must be have been built up contiguously reaching from one formeret arch to the next, which reached out into the two neighboring bays. Above the summits of the formerets, the whole vault masonry must then be built up together (this may be achieved by a small group of masons moving around). Such building procedure "springer by springer" instead of "bay by bay" must have been common to the vault constructions in the succession of Peter Parler's vault designs, hence, in most of the late Gothic vaults in the German speaking area. In fact, already the vault of the High Choir in St. Vitus Cathedral could be brought up only "springer by springer" until the summits of the formerets, and then in continuity over the full length. Current research is expected to bring more clarity on the design principles of these challenging constructions.

Fig. 13. Also in the higher portions the continuity over the whole squinch can easily maintained. Over the transversal cap, the courses are seamed (D. Wendland)

Fig. 14. At the intersection points in the groins, the direction of the masonry courses change. Never-theless, the continuity of the texture is maintained (D. Wendland)

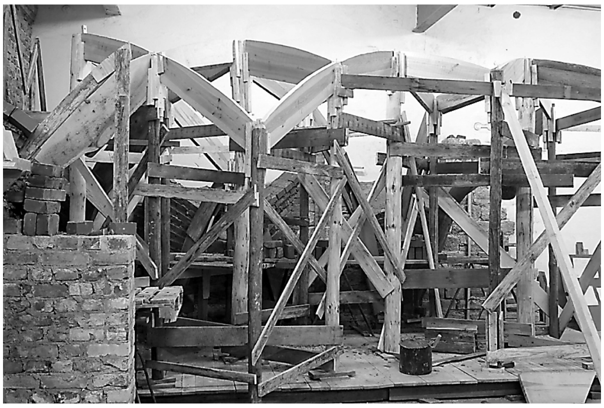

Fig. 15. Reconstruction of the centering used for building late Gothic "Cell Vaults", developed from surveys in correlation with sources and the experience of the experimental reconstruction (D. Wendland)

3. CONCLUSIONS

In the presented study, a methodology of analyzing historical construction is proposed, which enables to formulate statements on construction processes and design criteria in historical structures that lack documentation. This may be relevant in conservation or restoration by providing better knowledge of complex structures, as well as in basic research on construction history. The approach combines 3-D-surveying with geometric analysis on one hand, and experimental archaeology on the other hand.

Regarding the "Cell Vaults", which represent a peculiar, yet wide spread type of late Gotic vault construction, in any case emblematic for the complexity of vaulted ceilings from that period, the construction procedure could be clarified. The hypotheses developed upon detailed analyses from the archaeological evidence are verified in practice in the full scale experiments that were carried out. Further, information on the geometric features and on the construction details is provided in a new quality. This is a necessary precondition for the conservation of these structures.

The improved knowledge of the construction principles and details illuminates the original construction principles of late Gothic vault construction, which until now have been perceived through the optics of modern interpretation as established in the 19th century. Finally, through the collaboration with an academy of historical craftsmanship, this knowledge is disseminated to the technicians and craftsmen operating in the practice of restoration.

ACKNOWLEDGEMENTS

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Abstract

The so-called "Cell Vaults", also called "Diamond Vaults", are a particular type of late Gothic vaults, which appeared for the first time in Saxony and became common in central and eastern Europe during the late 15th and 16th century. They can be found in such prominent buildings as the Albrechtsburg palace in Meissen (Germany) or St. Mary's church at Gdánsk (Poland).

The reason for the invention and the popularity of these vaults is usually seen in the building process, which is believed to have been an improvement respect to usual Gothic rib vaults. However, this hypothesis is difficult to prove, because there is no information about the building process in the sources. In any case, the understanding of the building process and the construction principles is a core issue for explaining the peculiar shape of these vaults.

In the current research, attempts are made to formulate new, well-founded hypotheses on the building process and a better understanding of their construction and design principles. This is done by correlating detailed surveys and geometric analyses of vaults, comprising the curves, surfaces and especially the masonry texture, with experiments on the construction procedure and principles, which are carried out in full scale using realistic materials.

Hence, one focus is the methodology, discussing possible ways to determine the design and construction principles from the existing building, searching "traces" of its making. The other is exploring the particular late mediaeval masonry technique, essentially different from the modern, in practice.

Streszczenie

„Sklepienia komórkowe", znane również jako „sklepienia kryształowe", są szczególnym typem późnogotyckich sklepień, który pojawił się po raz pierwszy w Saksonii, a u schyłku XV i w XVI wieku został rozpowszechniony także w Europie centralnej i wschodniej. Sklepienia te występują w tak ważnych budynkach jak pałac Albrechtsburg w Miśni (Niemcy) czy kościół mariacki w Gdańsku (Polska).

Przyczyną wynalezienia i rozpropagowania tych sklepień jest proces budowlany, znacznie ulepszony w stosunku do zwykłych gotyckich sklepień żebrowych. Jednakże trudno tą hipotezę udowodnić, gdyż brakuje informacji źródłowych dotyczących procesu konstrukcyjnego. W każdym razie, zrozumienie procesu budowlanego i reguł konstrukcyjnych jest podstawą niezbędną do wyjaśnienia niezwykłego kształtu tych sklepień.

W opisanych tu badaniach podjęto próby sformułowania nowych, dobrze uzasadnionych hipotez na temat procesu budowlanego tych sklepień, oraz lepszego zrozumienia zasad ich konstrukcji i projektowania. Osiągnięto to przez skorelowanie szczegółowych ekspertyz i geometrycznych analiz sklepień, obejmujących krzywe, powierzchnie, a zwłaszcza fakturę elementów murowanych, z eksperymentami dotyczącymi procedur i reguł konstrukcyjnych wykonanymi w pełnej skali z wykorzystaniem pierwotnych materiałów.

Stąd też, z jednej strony zwrócono uwagę na metodologię, rozważono możliwe sposoby zidentyfikowania zasad projektowania i konstrukcji na podstawie istniejącego budynku, poszukiwano "śladów" jego konstruowania. Z drugiej zaś, na zgłębiono tę szczególną technikę murarską z okresu późnego średniowiecza, tak zasadniczo różną od stosowanej obecnie.

Ivan Giongo[1], Maurizio Piazza[2], Roberto Tomasi[3]

Cambering of timber composite beams by means of screw fasteners

Wyginanie drewnianych belek kompozytowych z użyciem łączników śrubowych

Keywords: Timber camber beam, Wood camber by screws, Reinforced wooden floors

Słowa kluczowe: drewniana stropnica łukowa, wyginanie drewna za pomocą śrub, wzmocnione podłogi drewniane

1. INTRODUCTION

When rehabilitating historical masonry buildings it is certainly not rare to come to deal with sagged timber floors which cannot be buttressed due to heritage issues. A similar problem occurs when historical buildings are readapted to a new building usage which provides for an increase in floor loads. In this case the timber floors, originally designed to bear low loads, will inevitably show an excessive midspan deflection (serviceability limit state). Therefore the development of a procedure which enables to "lift" a beam by just superposing a "dry reinforcement element", could prove of some interest.

If one considers a composite beam, as in Fig. 1a, where the fasteners forms a 90° angle with the beam axis, it can be seen that without any other external load all the compression forces due to the pressure generated by the screws are in equilibrium and therefore the beam remains undeformed. As soon as a load is applied Fig. 1b, the beam begins to sag and the two component elements exchange a system of forces similar to that in Fig. 1c. On the other hand, if the screws are positioned as in Fig. 1d, in order to reach the equilibrium, the two contact surfaces have to exchange a shear action (Fig. 1e) that is opposite to that in Fig. 1c and consequently the beam rises.

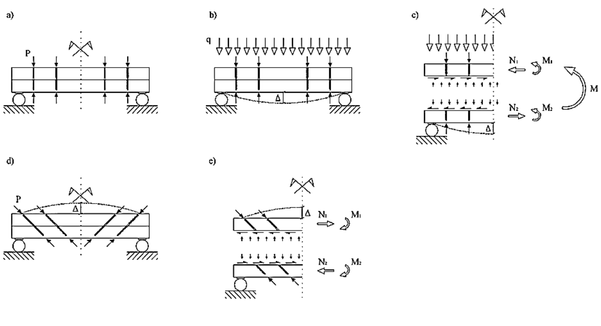

Fig. 1. Cambering principles for a composite beam

2. THE EXPERIMENTAL TESTS

The aim of this paper is to investigate the possibility of cambering a timber beam by simply putting another beam on the top of it and inserting screws inclined at 45° relative to the beam axis. So as to discover it, three tests have been carried out at the Laboratory of the Department of Mechanical and Structural Engineering (DIMS) of the University of Trento. Each specimen is composed by two 0,1 × 0,1 m^2 glulam beams 4 m long, connected by double threaded screws (Fig. 1). The fastener spacing (100 mm), is related to the need of obtaining a clear camber (more than 10 mm) through the connectors at disposal. It is utterly acknowledged that the flexural stiffness of a composite beam is directly related to the fasteners capability of hindering the two contact surfaces from slipping each other. Since the interface slip is maximum at the ends of the composite beam and minimum in the central part, cambering is expected to be more difficult when the screw assembly starts from the outer parts of the beam rather than when it

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starts from the inner part. Consequently tests No. 1 and 3 have been performed inserting the screws from the middle to the ends (Int-to-Ext) and test No. 2 has been carried out from the ends to the middle (Ext-to-Int). Before inserting any mechanical connector, a series of elastic bending tests has been performed in order to determine the MoE of the considered elements (Table 1).

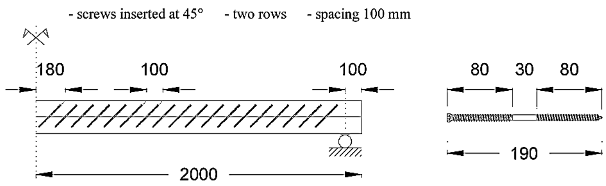

Fig. 2. Test setup

Table 1. MoE of tested elements

Composite Beam	C1		C2		C3	
Element	M1	M2	M3	M4	M5	M6
E [Mpa]	7327	12024	11863	8712	11358	9245

Table 2. Experimental upward camber

	wL/2 [mm]	Screwing pattern
C1	13.39	Int-to-Ext
C2	6.94	Ext-to-Int
C3	14.92	Int-to-Ext

Table 2 shows the results of the cambering procedure. As expected, test No. 2 (Ext-to-Int) exhibits a final value significantly lower than the other tests.

The camber amount (it has been observed an upward deflection of about one three-hundredth of the total beam length) could possibly be increased by reducing the screw spacing or by using fasteners able to generate a greater pressure. In doing so, keen attention should be paid to the magnitude of the internal stress state imposed by the cambering procedure. It is also quite evident that further testing is needed so as to fully understand the behaviour of such a composite beam in the long-term period. For the time being, the three assembled specimens have been monitored for 48 hours, during which no camber loss has been detected.

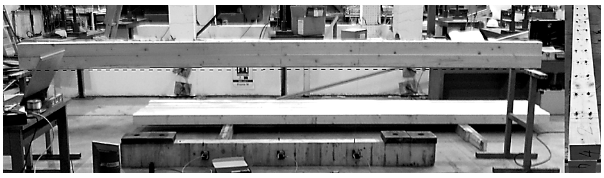
Fig. 3. Composite beam C1 after fastener insertion (starting from the beam centre)

3. THE NUMERICAL MODEL

A numerical model has been developed through the finite element software SAP2000. In particular, as to reproduce the act of inserting the screws one after the other, the nonlinear staged-construction function has been employed [1]. The choice of not utilizing the structure symmetry is due to the impossibility, during "real" assembly, of inserting the fasteners on symmetric positions simultaneously. However in that case, a slightly lower value of the final camber would have been reached since at the application of the screw pressure, the connector stiffness is already in place (other solutions have been tested but have led to excessive values of upward camber). Both the fasteners and the wood elements have been modelled as linear elastic materials. The stiffness of the screw couple Kc has been determined in accordance with [2] (Kc = 26303 N/mm) and has been reproduced by means of two crossed rods (inclined at 45°) whose axial stiffness is equal to Kc itself. The screw pressure has been introduced as a system of two inclined forces acting at the screw nodes. In addition, inextensible rods have been used to keep locked the distance between the barycentre lines of the wood elements.

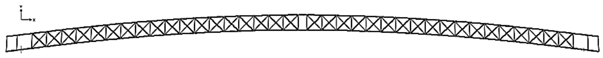
Fig. 4. The F.E. model

So as to determine what sort of pressure is to be assigned to the screw couple, some tests have been performed, relying on the setup showed in Fig. 5. Many parameters have been pried (e.g. screw angle with respect to the grain direction, initial pressure, head penetration length, threaded part length, connector typology, wood density, time-dependence) and further testing has already been started. A resultant pressure value of 4.4 kN for the single screw has been deemed as acceptable.

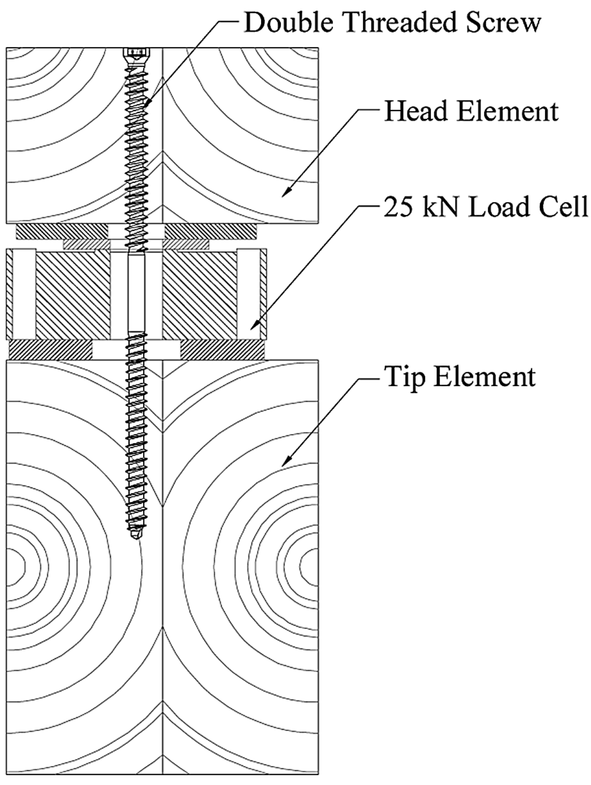

Fig. 5. Screw pressure test setup

The results obtained from the numerical model are given in Table 3 Regarding specimens C1 and C3, it could be seen that the numerical model reproduces the experimental behaviour with sufficient precision for both the tested screwing patterns (Int-to-Ext and Ext-to-Int). An underestimation of the camber value has been observed for specimen C3.

Table 3. Experimental data Vs. Numerical values [mm]

	Experimental	Numerical	Err. %
C1	13.39	13.78	2.91
C2	6.94	7.40	6.63
C3	14.92	12.52	16.09

4. THE ANALYTICAL FORMULA

Starting from equilibrium considerations [3] and taking into account the static scheme of Fig. 6, if one assumes the absence of the external bending moment, it is possible to obtain Eq. (1) and consequently Eq. (2):

$$N_{1,i}''(x) - \alpha^2 N_{1,i}(x) = 0, \ with \ N_{1,i}''(0) = 0, \ N_{1,i}(si) = T \qquad (1)$$

$$N_{1,i}(x) = \frac{T\sinh(\alpha x)}{\sinh(\alpha si)} \qquad (2)$$

where:

- $N_{1,i}$ is the axial force in the upper element of the composite structure (the beam length is equal to *si*);
- $N_{1,i}''$ is the second derivative of $N_{1,i}$;
- i is the number of the screw couple (labelling starts from the internal side);
- T is the horizontal component of the resultant pressure yielded by one couple of inclined screws;
- k_c is the distributed stiffness of fasteners;
- a is the distance between the centreline of the two elements;
- $\alpha = [(k_c EJ_\infty)/(EJ_0 EA_0)]^{0.5}$;
- EJ_0 is the flexural stiffness of the composite beam with no mechanical connections;
- EJ_∞ is the flexural stiffness of the ideal composite beam;
- $EA_0 = (\sum 1/EA_j)-1$;
- EA_j is the axial stiffness of the *j-th* element;
- s is the fastener spacing.

After determining $N_{1,i}$, the calculation of the *i-th* beam deflection can be attained as follows:

$$w_i''(x) = -\frac{N_{1,i}(x)\times a}{EJ_0}, \ with \ w_i(0) = 0, w_i'(0) = 0 \qquad (3)$$

$$w_i(x) = \frac{Ta}{EJ_0\alpha\sinh(\alpha si)} \times \left[x - \frac{\sinh(\alpha x)}{\alpha}\right] \qquad (4)$$

Hence it is easy to characterize the contribution of the *i-th* screw couple to the beam camber:

$$\Delta w_{i,L/2} = w_i(si) + w_i'(si) \times \left(\frac{L}{2} - si\right) \qquad (5)$$

Finally the evaluation of the beam camber is presented:

$$w_{L/2} = \sum_{i=1}^{n} \Delta w_{i,L/2} = \sum_{i=1}^{n} \left\{ \frac{1}{2} \times \frac{Ta\left[\cosh(\alpha si)\left(2si - L\right) + L\right]}{\sinh(\alpha si) EJ_0 \alpha} \right\} - \frac{nTa}{\alpha^2 EJ_0} \qquad (6)$$

where n is the total number (Fig. 7) of fastener couples.

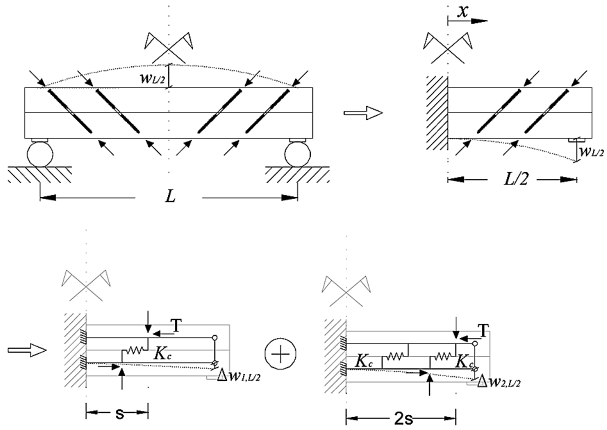

Fig. 6. Static scheme adopted for the analytical formulation

Table 4 provides a comparison between experimental data and analytical values obtained through eq. (6). The proposed formula seems able to reproduce the experimental camber of C1 specimen with quite good precision, while a certain error (13%) has been observed for C3 specimen. It should be noted that for composite beam C3 the numerical model gave a very similar prediction (19% err.).

Table 4 .Experimental data Vs. Analytical values [mm]

	Experimental	Analytical	Err. %
C1	13.39	13.28	0.82
C2	6.94	-	-
C3	14.92	12.88	13.68

As outlined in Fig. 7a, the effectiveness of an *i-th* screw couple depends on how many couples have already been inserted and on the fastener spacing. Although it has been

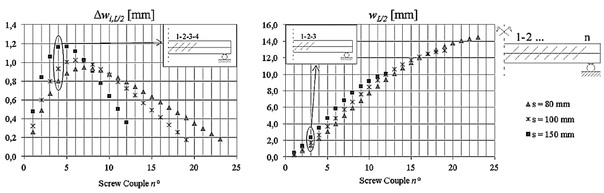

Fig. 7. a) Couple effectiveness to the upward camber; b) Camber evolution

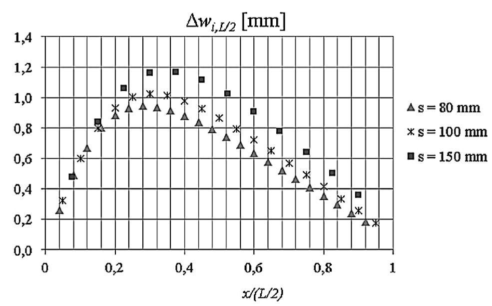

Fig. 8. Screw couple effectiveness Vs. Its position along the beam

observed that (Fig.8) the greater the spacing the greater the effectiveness, if one focuses on the global result it is clear that increasing the spacing reduces the amount of screws and consequently the final camber (Fig. 7b).

5. CONCLUSIONS

The exposed cambering procedure has proved to be effective and permits to obtain significant values of upward deflection. Obviously, the camber has to be consistent with what is connected to beam. In addition, it ought to be underlined that the experimental tests presented in the paper, have involved new timber beams with clearly defined boundary conditions. To assess the real effectiveness of this method (regarding the refurbishment of old floors), an experimental campaign on existing sagged beams, should therefore be undertaken. Particular attention will have to be paid to the internal forces that this procedure generates into an allegedly deteriorated beam.

Both the experimental tests and the numerical model have shown that the best way to obtain an upward deflection is to start the assembly from the centre and alternatively proceed towards the ends of the beam.

The proposed analytical formula seems to be able to reproduce the experimental behaviour and presents the benefit of being "easily manageable". This is mainly due to the choice of considering a constant fastener spacing along the beam axis. Otherwise, it would have been necessary to introduce Fourier transforms [4] that would have prevented the analytical model from being handled without a specific software for symbolic calculation.

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Abstract

The aim of this paper is to investigate the possibility of cambering a timber beam by simply putting another beam on the top of it and inserting screws inclined at *45*° relative to the beam axis. To this purpose, three experimental tests have been performed at the Laboratory of the Department of Mechanical and Structural Engineering (DIMS) of the University of Trento. After the calibration of a numerical model that helped in understanding the "cambering phenomenon", an analytical formulation has been proposed. The resulting formula for determining the upward camber (given the mechanical properties of the beams and of the fasteners) has shown the capability of reproducing the experimental behaviour with promising accuracy. However, further testing is recommended so as to validate the method feasibility to existing beams.

Streszczenie

Niniejsza praca ma na celu zbadanie możliwości wyginania drewnianej belki przez położenie na niej innej belki i zamocowanie śrubami pochylonymi pod kątem *45*° w stosunku do osi belki. W tym celu przeprowadzono trzy eksperymentalne testy w Laboratorium Wydziału Inżynierii Mechanicznej i Strukturalnej (DIMS) Uniwersytetu w Trento. Po skalibrowaniu modelu numerycznego, który pomógł lepiej zrozumieć "zjawiska wyginania", zaproponowano zapis analityczny. Otrzymana formuła na wyznaczanie wypukłości łuku (biorąca pod uwagę mechaniczne własności belek i elementów mocujących) wykazała zdolność powtórzenia zachowania eksperymentalnego z obiecującą dokładnością. Rekomendowane są dalsze testy aby sprawdzić możliwość zastosowania tej metody w odniesieniu do istniejących belek.

Elisa Poletti[1], Graça Vasconcelos[2]

Seismic behaviour of traditional half-timbered walls: cyclic tests and strengthening solutions

Zachowanie sejsmiczne tradycyjnych drewnianych ścian szkieletowych: badania cykliczne i rozwiązania wzmacniające

Keywords: Half-timber, Cyclic test, Traditional reinforcement, Dissipation of energy, Ductility

Słowa kluczowe: konstrukcja szkieletowa, badania cykliczne, wzmocnienia tradycyjne, rozproszenie energii, ciągliwość

1. INTRODUCTION

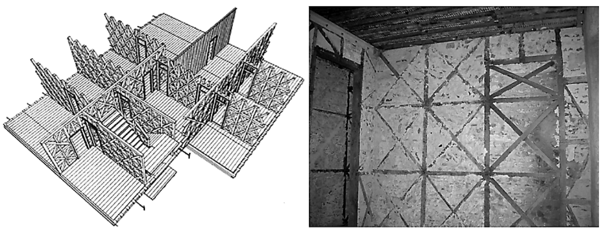
Half-timbered buildings have been a popular constructive system in many countries over the centuries. Masonry and timber are two of the most ancient materials used in construction and are easily available. The diffusion of these buildings makes their preservation of essential importance. The aim of this paper is to study the performance of half-timbered walls in their original condition and propose strengthening techniques.

1.1. Extension of half-timbered construction and historical importance

The origin of half-timbered structures probably goes back to the Roman Empire, as in archaeological sites half-timber houses were found and were referred to as Opus Craticium by Vitruvius [1]. But timber was used in masonry walls even in previous cultures (Mycean culture, Bronze Age) [2].

Traditionally, this type of structures was introduced as a seismic-resistant building. After severe earthquakes partially destroyed cities in the Mediterranean area (Lefkada, Reggio Calabria, Lisbon, Istanbul), new regulations [3-5] were introduced, which dictated how the new buildings should be built, introducing a bracing timber structure. But such buildings can be also found in non-seismic zones (UK, Scandinavia, Germany), due to the easily available materials that they adopt. Here too the buildings exhibit a timber frame, though the bracing members are less regular.

The example that is of most interest in this study is that of the reconstruction of Lisbon Downtown after the 1755 earthquake which destroyed that part of the city. The new regulations for the reconstruction of the city introduced by Marques de Pombal included a building of usually five storeys with a stone masonry ground floor and an internal timber frame structure (named gaiola in Portuguese, which means cage) for the upper floors (Fig. 1a). The gaiola was linked to the external masonry walls through the timber floor beams, to which it was connected. A minimal timber skeleton was present also in the external masonry walls. The framing of the gaiola was characterized by the typical St. Andrew's crosses (Fig. 1b), which provided a bracing effect to the structure. The walls were filled with rubble or brick masonry. The internal half-timbered walls originally did not participate in the bearing of the vertical loads of the structure, the load bearing walls were the external masonry ones, but subsequent alterations or changes in use of the structure could have altered this condition.



a) b)

Fig. 1. Examples of gaiola pombalina: (a) general floor plan [5]; (b) detail of half-timbered wall [6]

The types of connections and the dimensions of the cross sections of the elements varied, depending on the period in which they were built and the practice of the carpenter. In general, overlapped, dovetail, or simple contact connections were used between two elements, with the addition of nails to

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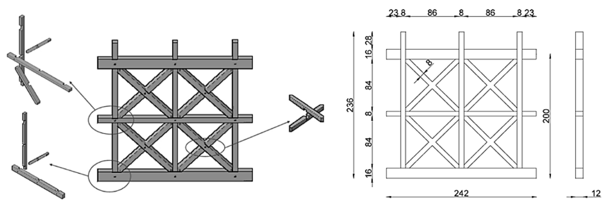
secure them in place [7]. Cross sections varied between 8 × 10 cm, 10 × 12 cm and 15 × 12 cm. Approximately a hundred years after their introduction, Pombalino buildings evolved to Gaioleiro ones, which lost the internal timber skeleton.

2. TESTS

2.1. Wall specimens and types of strengthening

Half-timbered wall specimens were prepared according to dimensions found in existing buildings in Lisbon. All the connections between the vertical posts and the beams are overlapped ones, as well as the connections between the two diagonals of the St. Andrew's crosses, whilst the connections between the diagonal and the main frame are simple contact ones (see Fig. 2a).



a) b)

Fig. 2. Wall specimens: (a) connections used; (b) dimensions of elements in cm

The walls were built in real scale, with realistic cross sections for all the elements (see Fig. 2b). The walls were first tested in an unreinforced condition, and subsequently they were retrofitted and strengthened with two types of strengthening: 1) bolts were inserted in each of the overlapped connections between the main vertical posts and beams (Fig. 3a) and 2) steel plates were applied to all the main connections between the main posts and the beams on both sides, taking into account also the diagonal member attached, and they were secured with bolts (Fig. 3b).

2.2. Test setup and instrumentation

Cyclic tests were performed on half-timbered walls using a reaction wall to which a hydraulic actuator was attached, which applied the horizontal displacement to the walls (Fig. 4). The actuator was connected to the reaction wall and to the top beam through two-dimensional hinges that allowed vertical displacement and rotation of the top border of the wall. Three hydraulic jacks applied the constant vertical pre-compression on the posts and could follow the horizontal movement of the walls by means of rods attached to the top of the jacks and connected to hinges fixed at the bottom steel beam. The walls were restrained at the bottom using steel angles and plates that fixed the bottom beam of the walls to a steel beam which was connected to the reaction floor.

Out-of-plane movements were prevented by means of steel rollers attached to an external frame securing the top beam of the walls.

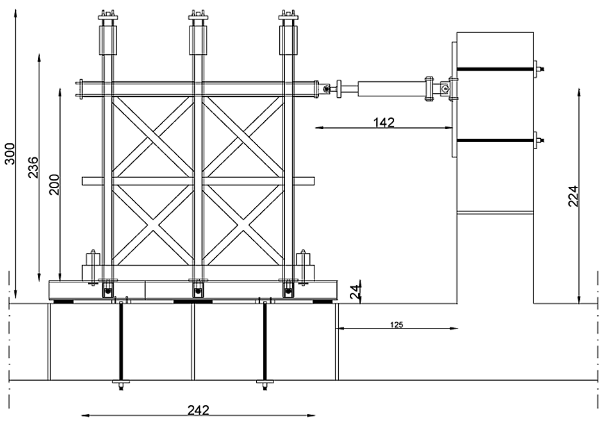


Fig. 4. Test setup used in the experimental campaign

Two different vertical loads were applied to the wall, namely 25 and 50 kN on each post. The application of different vertical load levels is significant, since the timber frame originally was not counted for the bearing of the vertical loads of the buildings, being their main function that of absorbing shear loads. But with modifications done to the structures, load redistributions could have occurred and additional loads could be present, which could be taken by the half-timbered walls.

2.3. Test procedure

A cyclic test procedure was adopted following standard ISO DIS 21581 [8], adding more steps in the procedure in order to better capture the highly non-linear behaviour of the walls. Due to limitations of the test equipment, the cycles were introduced with a sinusoidal law (Fig. 5), but no significant alterations were found in the tests when compared to others performed previously with linear cycles.

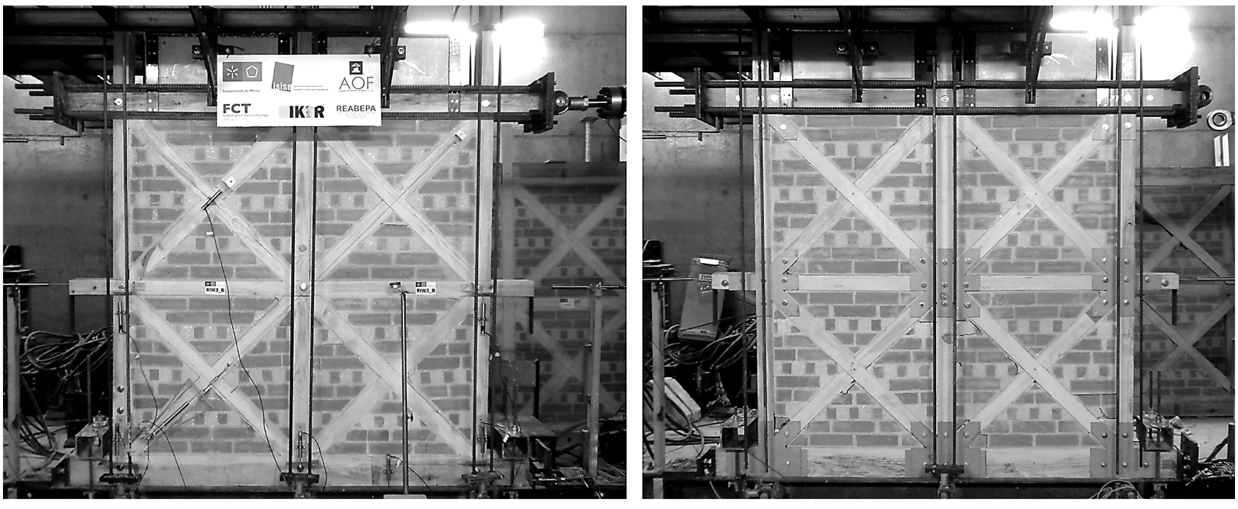


Fig. 3. Strengthening adopted: (a) bolts; (b) steel plates

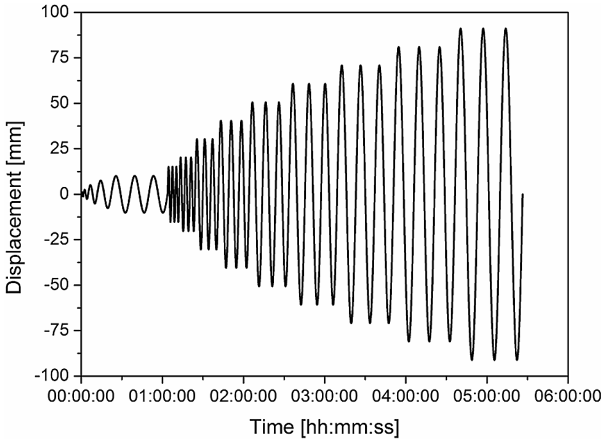


Fig. 5. Cyclic procedure adopted for the tests

Two different test speeds were adopted: for displacements up to 10% of the maximum one an average speed of 0,05mm/s was used; for higher displacements, a speed of 0,35 mm/s was adopted. The cyclic tests did not reach the ultimate displacement attained during the monotonic test (101,34 mm), but only 90% of this displacement, but it proved to be sufficient for the wall to fail under cyclic loading.

3. CYCLIC TESTS

Static cyclic tests can simulate in a simple way the seismic loading and provide important information on the overall mechanical behaviour and shear resistance of walls subjected to seismic actions. Cyclic test results performed on both unreinforced and strengthened walls are here presented and a discussion of their general behaviour is reported.

3.1. Results on unreinforced walls

Half-timbered walls were first tested in the unreinforced condition, subjecting them to an horizontal displacement proportional to the maximum horizontal displacement capacity of the wall achieved in a monotonic test, which resulted comparable to other results obtained performing similar tests [9].

A distinctively different behaviour can be noticed with a varying vertical pre-compression. The typical S-shape curve of a flexural response with a rocking mechanism at the bottom of the wall is evident for the walls subjected to a lower vertical load level (Fig. 6a), whilst a predominant shear behaviour is encountered in walls under a higher vertical load level (Fig. 6b).

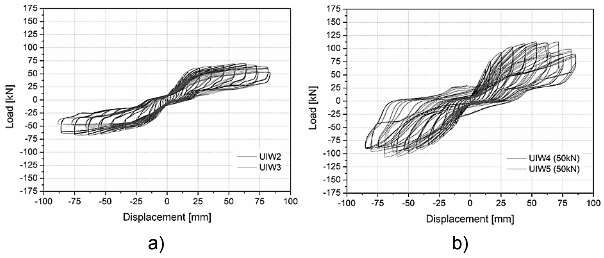


Fig. 6. Hysteretic curves for unreinforced walls: (a) lower vertical load level: (b) higher vertical load level

The walls exhibited, for low vertical load levels, a highly predominant flexural behaviour. They reach the maximum load level at a horizontal displacement of approximately 50mm and the subsequent loss of capacity is 20% of the maximum load or less. Due to the connection type, the vertical posts uplift during the test and the wall rocks back and forth. The lateral posts uplift as much as 50 mm (Fig. 7a).

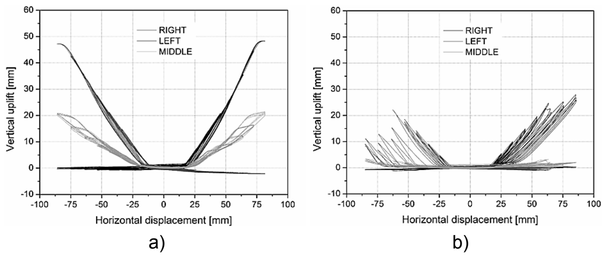
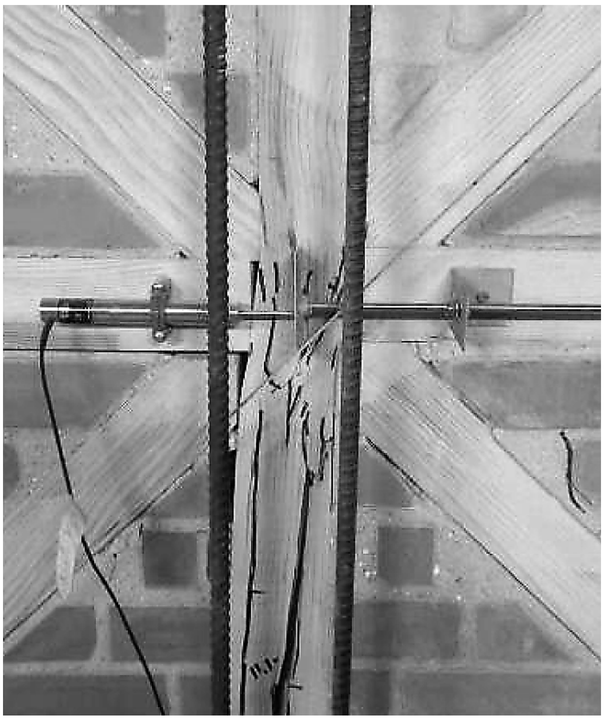


Fig. 7. Vertical uplift of posts for right, left and middle connection in unreinforced wall specimens: (a) lower vertical load level; (b) higher vertical load level

When a higher vertical pre-compression level is applied, the domineering mechanism is shear, but the flexural mechanism is still present, as it can be seen from the amount

a) UIW3



b) UIW5

Fig. 8. Uplifting of posts and opening of connections

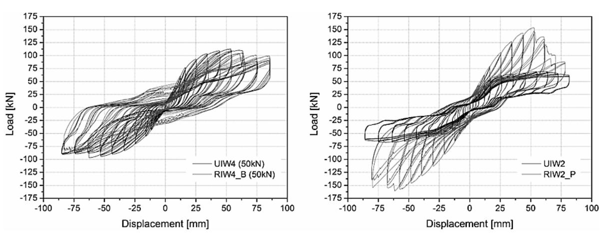
of vertical uplift of the lateral posts, which is lower (42%) if compared to the wall subjected to a lower vertical load, but it still characterizes the response of the wall. The walls tested under a higher vertical load level reach their maximum capacity at approximately the same horizontal displacement, but the post peak loss of capacity is higher (33% for UIW4 and 22% for UIW5). When observing the hysteretic curves of the walls (Fig. 6), the change in stiffness that can be observed in the unloading path occurs when the vertical uplift of the bottom connections returns to be zero, thus increasing the stiffness of the wall, as more resistance is given by the posts.

For the lower vertical load, the walls do not present significant damage, while it is present for the walls tested under the higher load. In fact, while the walls subjected to the lower level fail due to the opening of the bottom connections, with the vertical posts uplifting at the bottom (Fig. 8a), the higher loaded walls fail due to shear in the central connection (Fig. 8b) with wood crushing. The bottom connections tend to open and uplift in this case too, but they don't control the general wall failure.

Concerning the infill behaviour, masonry behaved like a block. Few fissures could be observed, mainly mortar falling out at corners and corner bricks falling out due to nails tearing-off. A few compression cracks were visible. The blocks of masonry tended to move out-of-plane, as the adhesion of masonry to timber is very low and when the elements uplifted the masonry would detach and move out. After the test, it was almost always possible to push the masonry blocks back into place, so that masonry degradation did not influence the performance of the strengthened walls.

3.2. Results on strengthened walls

The tested walls were retrofitted, strengthened and tested again. As stated previously, the damage to masonry was recovered completely. The same cannot be said for the damage to the timber to timber connections. The nails tore the wood and, in the case of the walls subjected to the higher load, the vertical element of the central connection had to be substituted, introducing a new element to the post which was glued with a structural glue in order to guarantee the continuity of the element.



a) b)

Fig. 9. Hysteresis curves of strengthened walls: (a) strengthening with bolts (higher vertical load level); (b) strengthening with plates (lower vertical load level)

In the case of strengthening done with bolts, the improvement in terms of load capacity, energy dissipation and ductility is not overly significant. For the lower vertical load level, the retrofitted wall experienced a gain in terms of load capacity of 24%, while for the wall subjected to a higher vertical level there was no gain (Fig. 9a), but the wall regained its initial capacity and improved in terms of energy dissipation. The insertion of bolts did not influence the overall behaviour of the walls. The response of the wall is characterized by a combination of flexural and shear mechanisms, the posts continue to uplift, though the vertical uplift decreased of 40% when compared to the unreinforced condition (Fig. 10a). The main advantage is that the connections were now unable to open out-of-plane, thus allowing them to function until the ultimate displacement, whereas in the unreinforced specimen, the opening of the connection effectively caused that connection to cease working properly. For wall RIW4_B, the reduction of the vertical uplift (Fig. 10b) was less (30% when compared to UIW4).

Thanks to the bolts, the connections worked properly and a shear failure was obtained even for the specimen subjected to a lower vertical load level, with the central beam tearing due to the shear caused by the diagonal elements.

For the wall specimen subjected to a higher vertical load level (RIW4_B), the failure occurred due to the crushing of the central connection caused by shear as well as the tearing off of the lateral central connection (Fig. 11a) which caused the wall to open in plane and reduced the stiffness of the wall in the unloading branch, since the left post was not participating fully to the reaction of the wall when the top beam was being pulled.

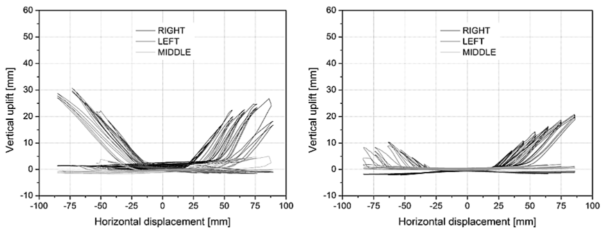
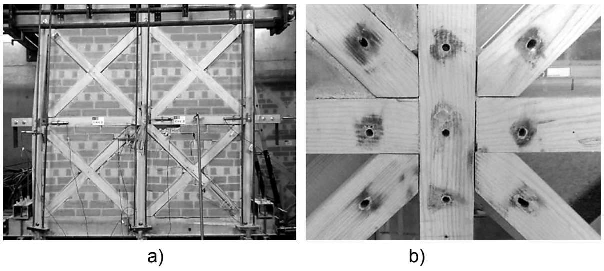


Fig. 10. Vertical uplift of right, middle and left posts for walls strengthened with bolts: (a) lower vertical load level; (b) higher vertical load level

Analysing the behaviour of the walls strengthened with steel plates, it can be observed how this type of strengthening highly stiffens the wall (Fig. 9b). The gain in load capacity for the wall subjected to the lower vertical load level was of 121% when compared to the unreinforced condition. RIW2_P failed due to failure of the bottom corner connection because the plate did not allow the post to uplift and the post tore in correspondence of the bolt.

The plates did not show great deformations, but the holes of the bolts generally deformed becoming oval, especially the ones corresponding to the diagonal elements of the central connection (Fig. 11b), which are the elements that work more as they push and pull the central connection, and those of the bottom connections, since those were the ones which normally tended to uplift more.

Wall RIW5_P showed a gain in terms of load capacity of 36% after an applied displacement of 50mm. The test was not completed due to problems in the equipment, but it was noticed that, even for the higher vertical load level, the strengthening with plates is highly efficient in terms of load capacity.



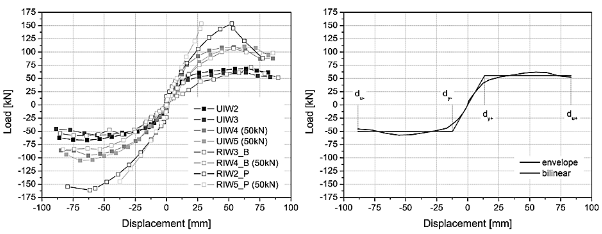
a) b)

Fig. 11. Typical damages in strengthened walls: (a) shear failure in RIW4_B; (b) holes in RIW2_P deformed becoming oval

4. SEISMIC PARAMETERS

The study of the seismic behaviour of a structure is essential when designing a new one or rehabilitating an existing one. Various parameters, such as ductility, energy dissipation, cyclic stiffness, equivalent viscous damping, characterize the behaviour of timber shear walls and can help in evaluating the performance of a structure under cyclic loading. Here, a few parameters are presented.

The envelope curves of the first cycle repetition of the hysteretic diagrams were obtained joining the points corresponding to the maximum load reached during each cycle at its respective displacement [8]. Fig. 12a shows the curves obtained in such a manner. From the envelope curves, the initial stiffness of the walls was obtained, considering the portion of the curve up to 40% of the maximum load to calculate the secant stiffness, as stated in [8]. Among the unreinforced walls, the vertical pre-compression level did not influence significantly the initial stiffness. All walls exhibited a similar stiffness, values varied between 4 and 4.5 kNmm, except for wall UIW5, which had an initial stiffness of 3kNmm, lower than the others because it experienced pinching from the early stages, due to important clearances in the connections. For the strengthened walls, strengthening done with bolts did not increase the initial stiffness, the walls were retrofitted but the connections had already suffered some damage from the tearing-off of the nails and the bolts could not recover the initial performance of the wall. In fact, for both vertical load levels, the initial stiffness decreased of almost 40% (the stiffness values were 1.89 and 2.59 kNmm for the lower and higher load level). For the plates strengthening, the walls showed an increase in initial stiffness of 50% (initial stiffness reached 6kNmm for both walls), pointing out how this method highly stiffens the wall, thus the significant gain in ultimate load, but reduces ductility, which from a seismic point of view is often more important than stiffness. Moreover, the initial stiffness of the two walls is very similar, possibly pointing out that for this type of strengthening, the vertical pre-compression level is not as significant as for the strengthening with bolts.



a) b)

Fig. 12. (a) Envelope curves of tested walls; (b) example of bilinear idealization of envelope curve

From an envelope curve it is possible to obtain a bilinear idealization of the same (Fig. 12b). Different methods can be used to obtain this idealization, for example the ones suggested in [10] or [11]; in this study the approach proposed by Tomazevic was used [11], i.e. the failure load was considered as 80% or more of the maximum one and the yield displacement and load were calculated from the equivalence of the areas underneath the curves considering the initial stiffness obtained from the envelope curves. From here the values of ductility were derived for the various walls tested. In general, the unreinforced walls presented a higher ductility and among these, the walls with a lower vertical pre-compression level had the highest ductility (an average ductility of 6.5 versus a ductility of 3.5 for the walls with higher load). Among the strengthened walls, the strengthening carried out with bolts did not improve the initial stiffness of the walls or their ductility. Nonetheless, the walls were able to perform well and reached the same load capacity of the original walls. The bolts strengthening appears to give a better performance in the post-peak, since it keeps the connections closed so that they can continue to work, which did not occur in the unreinforced situation. The plates strengthening gave similar results to the bolts one in terms of ductility; all strengthened walls had a ductility of 3.0.

The energy dissipated by the walls is computed at each load cycle by calculating the area enclosed by the loop in the load-displacement diagram. It represents the amount of energy dissipated during the cyclic loading which occurs through friction between joints, yielding of nails and non-recoverable deformation (residual deformation) in the wall panel.

The low values of dissipated energy (Fig. 13) for low levels of vertical pre-compression are associated to the predominant flexural rocking mechanism prevailing for this load level. Walls subjected to a higher vertical pre-compression (UIW4 and UIW5) present higher dissipated energy.

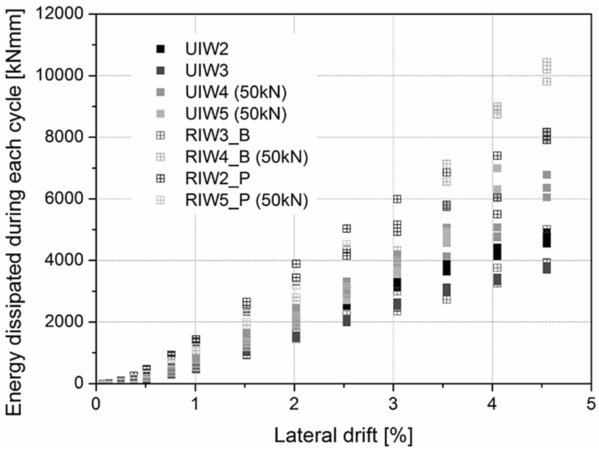


Fig. 13. Energy dissipated during each cycle vs. lateral drift of the walls

There isn't a significant improvement in terms of energy dissipation between the unreinforced and the wall strengthened with bolts. Nonetheless, since the walls were already tested and the strengthening solution considered was extremely simple and did not change much the original behaviour of the wall, it can be pointed out how with these few devices the walls were restored to their original conditions and improved their performance for higher values of drift. In the case of the walls strengthened with plates, for the lower vertical load level, which is the only case that has complete data, the gain in terms of dissipated energy is evident from low values of lateral drift, pointing out how the steel plates represent a more efficient strengthening even in terms of dissipated energy.

5. CONCLUSIONS

Static cyclic tests were performed on traditional half-timbered walls. The behaviour of unreinforced walls and retrofitted and strengthened walls was compared in order to understand their behaviour in seismic situations. In the unreinforced condition, a higher vertical load level led to a higher load capacity and energy dissipation. Damages to the timber frame were more important for higher vertical loads, whilst for the lower ones the damages concerned mainly some nails tear-off.

For the strengthened walls, bolts strengthening only managed to reinstate the walls to their initial condition, with some advantages in the post-peak behaviour in terms of resistance and energy dissipation. The plates strengthening proved to greatly stiffen the wall and increase considerably its load capacity, but ductility was compromised.

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Abstract

Half-timbered buildings represent an important historical heritage in many countries. They are diffused in various regions for different reasons, such as availability of materials, to lighten a structure, their low cost, the strength they offer and as a construction element able to resist seismic actions. This latter issue is the research topic analysed here, as half-timbered buildings have been specifically used in reconstruction plans as earthquake-resistant buildings in many countries, such as Portugal, Italy, and Greece. All these buildings were characterized by an internal timber skeleton constituted of vertical and horizontal elements and braced with diagonal elements (St. Andrew's crosses). This structure aimed at improving the global stability of masonry buildings, enhancing their capacity to dissipate energy during earthquakes.

The aim of this paper is to study the behaviour under cyclic loading of such half-timbered walls, with typical connections, materials and geometries encountered in existing buildings. In general, half-timbered walls act as shear walls and confer to the masonry structure a better seismic resistance than that provided by a traditional masonry wall. Cyclic tests were performed on traditional walls and their behaviour was studied in terms of ultimate capacity, deformability, energy dissipation and stiffness. Subsequently, the tested walls were retrofitted with traditional techniques in order to understand the influence of the reinforcement and to estimate its effectiveness, or lack thereof.

Streszczenie

Budynki o konstrukcji szkieletowej stanowią ważny element historycznego dziedzictwa w wielu krajach. Są rozproszone w rozmaitych regionach z różnych powodów, takich jak dostępność materiałów, zmniejszenie ciężaru konstrukcji, jej wytrzymałość, niskie koszty, oraz jako element konstrukcyjny zdolny wytrzymać wstrząsy sejsmiczne. Ta ostatnia kwestia jest przedmiotem niniejszej analizy, gdyż budynki o konstrukcji szkieletowej są wykorzystywane szczególnie w projektach rekonstrukcji w krajach, takich jak Portugalia, Włochy czy Grecja, jako odporne na trzęsienia ziemi. Wszystkie te budynki charakteryzuje obecność wewnętrznego szkieletu drewnianego składającego się z pionowych i poziomych elementów, połączonych elementami ukośnymi (krzyże św. Andrzeja). Taka konstrukcja ma na celu poprawę ogólnej stabilności budynków murowanych, zwiększając ich zdolność do rozpraszania energii w wypadku trzęsienia ziemi.

Celem niniejszej pracy jest przestudiowanie zachowań ścian szkieletowych wykonanych z użyciem typowych połączeń, materiałów i geometrii, spotykanych w istniejących budynkach, pod cyklicznym obciążeniem. Na ogół ściany szkieletowe odpowiadają za przenoszenie ścinanania i nadają konstrukcji murowanej lepszy opór sejsmiczny, niż ten jaki daje tradycyjna ściana murowana. Badania cykliczne zostały wykonane na tradycyjnych ścianach, a ich zachowanie przestudiowane pod kątem całkowitej nośności, odkształceń, rozproszenia energii i sztywności. Następnie testowane ściany zostały wzmocnione metodami tradycyjnymi, co miało pomóc w zrozumieniu wpływu takiego działania i ocenie jego efektywności, bądź jej braku.

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Conservation renovations of the bridges in the Muskau Park

Remonty konserwatorskie mostów w Parku Mużakowskim

Keywords: The Muskau Park, Bridges, Conservation

Słowa kluczowe: Park Mużakowski, mosty, konserwacja

1. HISTORY OF THE PARK

The Muskau Park, as it is known in the history of garden art, is considered to be an outstanding composition modelled on English parks, surprising one with the individuality and originality of its solutions. The park composition is an expression of the unhindered imagination, artistic licence and originality of its creator – the owner of the local estate Prince Hermann von Pückler-Muskau, a person who has gone down not only in the history of gardens, but also in the history of the European culture of the 19th century.

The park laid out by him in the valley of the Lusatian Neisse river (Fig. 1) has been recognized, both by his contemporaries and the posterity, as the purest and most original expression of Romanticism in garden art. In its composition the creator in a unique and ingenious way managed to exploit the scenic values of the land. By transforming the existing residence Pückler created, on the area of over 800 ha, not only a park composition, but also a new, unusual in its scale and expression, landscape designed in the minutest detail – a microworld understood as a poetic ideal which became part of the surrounding reality. A story, partly true and partly invented or adapted by Pückler, became its content. The layout, referred to as *Haupt-Idee* [1], was to be "nothing else, but only a desire to reflect a tender image of our family or the native aristocracy, who received their education here, so that this idea could develop, so to speak, on its own, in the observer's soul". In order to reflect this content, all what potentially was situated within the composition – both people and any evidence of the local history or objects deemed by Pückler to be such evidence – the past and the tradition – combined to form a whole, was to constitute together with the present, a visualization of the hitherto existing order.

Pückler devoted a fortune and the best years of his life to realize his work in the years 1811-1845. The large-scale investment programme ultimately led to a financial collapse and to the sale of the estate [2]. Although at the time of the sale the park reached the size comparable with the concept, extensive lands, particularly on the east side, have never been developed. Nevertheless, the original concept of the work, which he transferred into the realms of fantasy, was fully realized on the pages of a book [1].

Fig. 1. Panorama of the Parkfrom the east site (19th cent.) [1]

After it had been sold to the Counts von Nostitz and von Hatzfeld, in 1846 the estate became the property of Prince Willem Frederic Karel der Nederlanden (Willem Frederic Karel von Oranien, 1797-1881), the son of the later King of the Netherlands Wilhelm I. In the person of the prince the park gained not only an owner, but a caring patron, full of understanding and respect for his predecessor's work. After his death, from 1883 the next owner of the Muskau estate was Count Traugott Heinrich von Arnim (1839-1919), a diplomat and for many years Bismarck's personal secretary. Also the years 1883-1945, when the park remained in the hands of the successive generations of the von Arnim family, were a period of full respect for the achievements of the park creator. However, as a result of the progressing industrialization and the considerable deterioration in the economic situation of the estate, local transformations of the composition did take place in that period.

As the consequence of the new order in Europe after 1945 the park composition was cut in two by the border between two countries. The residential centre with the Castle Park, the Mountain Park and the Spa, a fragment amounting merely to one third of the composition (about 200 ha), found itself on

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the German side. Since the fragment included architectural structures it could function independently. Once the difficulties of the first post-war years had been overcome, already at the beginning of the 1950s the heaviest damage was repaired. In 1953 the park board was established and in 1955 the park was designated as a monument. Thus the formal basis for the protection and conservation of the park was created. The damaged buildings began to be conserved and in 1965 the full restoration of the Old Castle, which was to serve as museum, began. The Bachelor House was extended to serve as a sanatorium. Thus the use of the park on the German side was determined by the spa tradition, rooted in the history and being of great local importance. Owing to this through the whole post-war period the Park was maintained up to high, for those times, conservation standards. The east part of the Park – the former suburbs of a small German town, amounting to as much as two thirds of the Park's historic area (about 700 ha), found itself on the territory of Poland. This fragment, taken out of context, devoid of compositional reference, was administratively incorporated into the newly created town of Łęknica. The "tree-covered" part was incorporated into the State Forests National Forest Holding and the historic fancy farm was taken over by the State-Owned Farm. The Park would get overgrown and some of it was under forestry cultivation, whereby the interconnections between the two parts would become obscured over time.

Fortunately, from this condition the Park was brought back to life thanks to the Polish-German agreement on undertaking its restoration, and reconstructed jointly in 1991 as the first monument commemorating its creator, it became the foundation stone not only for the restoration of the Polish part of the park, but also for the whole process of reintegration of the historic composition.

2. PARK COMPOSITION

Pückler meant the park he designed to be a synthesis of different fields of art – "a holistic work" – *Gesamtkunstwerk*, in which the fields of art, jointly with nature, coexist on equal rights [1]. He pursued this aim in an exceptionally consistent and multiplanar way through all the elements of the programme.

The shape of the future composition was mainly determined by the location of the main buildings of the family residence, situated on the left bank of the Lusatian Neisse river, widening in this place of the valley, and by the unique lie of the land, created by the Lusatian Neisse when it broke through the geological formations of the frontal moraine formed during the Mid-Polish glaciation, today called Muskau Arch (Muskauer Faltenbogen). The formations were designated as a Scenic Park in 2001 and as a National Geopark in 2009.

The project planned on a grand scale involved a wide range of work connected with the necessary remodelling of the residence's main buildings, changing the routes of the surrounding streets, building new structures, also artificial lakes and streams, drainage work, changes in the topography and planting thousands of trees.

The centre of the park composition was the complex of three main buildings: the Old Castle, the New Castle and the theatre, deriving from the earlier layout. The buildings (except for the Old Castle) were designated by Pückler for general remodelling, which was the basis for the original concept of modernization of the whole layout, began by him after 1815 [3]. The idea consisted in not only changing the form of the buildings, but also in their full integration with the surroundings. Therefore it was essential to remove the old fortifications and the moat, including the retaining walls erected by his grandfather [2]. The artificial Lucie Lake (also called the Castle Pond) was created in their place. The lake waters surrounding the palace on its three sides provided a proper background for the building. The bridge in the east side entrance was replaced with a ramp.

The idea of the ramp was suggested to Pückler by John Adey Repton (1787-1849), a son of the famous English pianist Humphry Repton (1752-1818). Pückler invited the eminent Prussian architect Karl Friedrich Schinkel (1781-1841) to create the architectural designs. The ramp built according to his design was to be one of the few building plans ever put into practice by Pückler. The ideas of the general remodelling of all the three buildings and integrating them into a single architectural complex have never been implemented. According to the designs supplied by Schinkel the buildings were to be given Gothicizing forms and connected into one complex reflecting the many centuries of the history of the place. The former official residence was to be explicitly identified with the oldest element of this complex. Hence the name Old Castle to emphasizing this intention. The buildings were connected by two bridges supported by arcade arches, constructed on different levels, with a tower in the central part.

Main residence buildings were surrounded with richly equipped flower gardens constituting an extension of the dwelling interiors. Among the abundance of architectural and plant detail, parterres, original and fanciful in form, found application here. The central part of the residence was then surrounded by a pleasure ground – a park zone used for strolling, cut through by the scenically shaped Hermann Neisse– an artificial branch of the river, dug through in order to enrich the park sceneries. Owing to this, artificial lakes – the above mentioned Lucie Lake and the Oaks Lake in the north part of the Castle Park – were created. The Hermann Neisse river was accompanied by numerous little bridges and picturesquely shaped cascades and appropriately planned out park roads. As a transitional zone the pleasure ground, on one hand, included very decorative elements, making it resemble the gardens, and on the other hand, by the extensiveness of the park interiors and the freedom of its form it alluded to the composition of the park proper. It was characterized by quite extensive, exquisitely manicured lawns, rare plants and original in form shrubbery arrangements scattered on the lawns [5].

The rest of the park, the park proper, laid out above the town adjacent to the residence, and also, in major part, stretching on the east side of the Neisse, was clearly distinct from the central park composition. The river terraces rising above the valley, as high as 50 m, and selected places on their edges on both sides of the river were used to situate there structures viewed from the centre of the park and vice versa. Special structures, e.g. the gazebo in the Castle Park, and park benches situated in the places selected by the creator, were used to admire the views. Thus the valley's two sides were interconnected by a network of vantage points, however, still focusing attention on its interior in whose centre the "lordly residence" dominated [5]. The park filling the valley on both sides of the Neisse river was to create the impression of a gallery of pictures viewed according to a scenario constructed by Pückler, showing the main buildings of the residence, the pavilions, the structures and the park scenes in ever different frames and perspectives. In the pictures the structures, having not only a decorative character, but also performing useful functions, e.g. the bridges linking the edges of the gorges together, would blend in with the natural, or rather almost natural, sceneries. The desired effects would be produced through the skilful composition of plant elements, the sometimes slight modelling of the terrain, or simply the "retouching" of some parts. Often these were solely compositions of trees, arranged in groups of different size, clumps, or standing solitary against the background of a vast interior, laid out with dramatic tension characteristic of Pückler. A substantial role in those compositions would be played by venerable trees, especially oaks several hundred years old, attracting attention because of their great size and noble form.

The skilfully designed roads would lead the visitor from point to point, along the edges of the terraces, offering various impressions, depending on the time of the day and the lighting. The sunlit sequences of vast glades and the long open vistas contrasted with the shady parts, especially with the muted natural sceneries of the gorges, into which this time narrow paths would lead.

Among those vast spaces there was no lack of attractive destinations for expeditions. On the right bank of the river there was the Spa with curative waters and mud baths, in the northern regions of the east part of the park a charming hamlet – a complex of pavilions and equipment for games and play in the English cottage style, called "England" or the "English House" (Englisches House) was located.

On the pattern of the 18th century English gardens also farmlands were included in the composition. Adding variety to the extensive field areas, groups of trees and solitary trees formed a landscape composition in which, at least in the conception, the practical aspects were combined with the decorative ones.

The particular parts of the park were connected by green promenades, enclosing the town of Muskau and its agricultural suburbs within the composition [5].

3. BRIDGES, SMALL BRIDGES, VIADUCTS

3.1 Cross-border bridges

The architectural programme for the park was consistently defined by Pückler in a way which made it possible to implement the main idea. It comprised of a few tens of buildings and structures, representing a great variety of cubatures and architectural forms, but forming a clear and cogent complex correlated in terms of content. This programme described in [1], however, only to a small degree coincided with the programme actually carried out by Pückler.

Besides the architecture of the main buildings situated in the central part of the park, the diversity of the styles of the park structures was fully consistent with the principles of the picturesque style. Despite the partly commemorative character of their forms (e.g. the English House as a souvenir of a journey), through their location in specific spatial contexts the contents associated with them acquire new meanings, supporting the implementation of the historical programme assumptions. Most of the planned buildings and structures had also a practical character, which was an important postulate of the creator. Also the bridges represented a variety of forms. However, most of them in a later period were replaced with new structures or disappeared.

From among the bridgestone should single out two bridges over the Neisse river, situated in the central part of the composition, conditioning the functioning of the Park as a whole. The Double Bridge (Doppelbrücke), consisting of two segments separated by an island, connects the centre of the residence with the nearest park interiors on the east side. The other bridge – the English Bridge, situated north of it, constituted the shortest connection between the palace and the cottage (the English House) on the east side. Erected in 1822, as a wooden structure, it was destroyed by the waters of the Neisse several times, each time acquiring a slightly different form.

In 1867 the Double Bridge was replaced by a masonry structure with granite and clinker piers. Blown up during war operations, it was rebuilt as part of an agreement between the Polish and German Park Boards in 2003. The English Bridge, also erected as a wooden structure in 1822, gained in 1858, in the times of Frederic der Nederlanden, new masonry piers. The balustrade was changed several times, to be finally replaced, after the bridge was damaged by flood in 1897, with a wrought-iron one. The bridge was rebuilt in 2011, in its last known form dating back from before 1945.

Using the two bridges one could travel in the park along a ring road whereby one could get to know the extensiveness and uniqueness of the composition. Besides their communication importance the two bridges were also objects viewed from different perspectives.

3.2. Bridges on German side

In the Castle Park, on the park roads and paths accompanying the Hermann Neisse river, which in several places intersect its course, there were located several small bridges. Each of them was given an individual form, matched to the character of the composition of the particular area, and its own name. And so there is Carp's Bridge, formerly wooden, replaced by a small bridge with cast-iron balustrade in the 1860s, the Castle Bridge between the New Castle and the Bachelor House, with an original cast-iron balustrade, the wooden Rheder Bridge (Rhederbrücke), dedicated to the first gardener of the Park, the Shepherd Bridge (Schäferbrücke) and the brick-and-fieldstone Bridge by Oaks' Lake (Eichseebrückewasserfall).

The Red Bridge in the Mountain Park was built together with the Upper Road in about 1830. Situated above a small street leading from a small town (Bergsche Kirchgasse) to the Mountain Park, it is a red brickwork structure with a wide arch. Its openwork brick balustrade topped with a sandstone parapet was modified as a result of restoration work carried out in the 1960s. The original form of the bridge alluded to the wall near the gardener house by the Turkish Baths in the Sanssouci garden [2].

3.3. Bridges on Polish side

Three masonry bridges are the only historic structures on the east side of the Park, which survived World War II. All the three were erected after the middle of the 19th century, in the times when the Park was owned by Frederic der Nederlanden. The bridges, by design, have double importance. On one hand, connecting the edges of the gorges they ensure the continuity of the park roads and on the other hand, as local dominant features they are part of the park scenes observed in the views from the Park.

Prince's Bridge, also called the Royal Bridge in honour of prince and later king Frederic Wilhelm IV (1795-1861), is situated within the first above-flood-level terrace north of Pückler's Stone. Designed and erected by Pückler as a wooden structure, in the form of a trellis covered with grapevine, made from "young oaken rods in the form of arches", it is shown in plate XXV in [1].

The bridge was rebuilt in 1854. Since then it has been a brick structure supported by three arcades, with an openwork sandstone balustrade. Its current good condition is the result of restoration work carried out a few years ago.

The Arcade Bridge is situated within the second above-flood-level terrace, near the Mausoleum Terrace. It is a monumental structure supported by five arcades, connecting the edges of a deep gorge on the bottom of which Sara's path runs. Erected in 1853, it replaced the wooden bridge – the Viaduct over Sara's path – previously existing in this place. In the construction of this bridge, being at the same time a viaduct, besides brick, also glass slag, a byproduct of the smelting of iron in the 19th century ironworks, was used. The original form of the bridge is not known.

The Viaduct is a masonry bridge connecting the edges of a deep gorge cutting through the second above-flood-level terrace. The structure planned by Pückler was finally erected as late as after 1862/63. It was built from brick and glass slag, with an ogival gate opening. The original form brings to mind a gate inviting one to the interior of the park and it is regarded as such since the beginning of the 20th century. Overhead runs a road from Lord's Mountain (Herrenberg) towards the north end of the park. A source included in [1] shows a bridge supported by five arcades.

Moreover, in Pückler's times there also existed an Oak Footbridge – a small wooden bridge over the gorge on the Nightingale Path route. The bridge was built from tree trunks and brushwood. Mentioned in [1], shown in lithograph no. XXV in the collection appended to [1], it existed until 1865, as confirmed by cartographic materials and later accounts.

Neither the form of the Chain Bridge [1], probably suspended over the gorge with the Hermann Road leading directly to the dairy, is known.

4. PARK TODAY

The restoration of the park as a whole made up of two integrally interwoven parts is not merely a restoration problem. It is a process which required and still requires the solution of a whole range of administrative, legal and conservation problems.

Each of the park's parts has its own peculiarity stemming from the historical and compositional determinants and its condition.

In the German (west) part of the Park, since 1993 managed by the Saxony Prince Pückler Foundation in Bad Muskau (Stiftung Fürst-Pückler-Park Bad Muskau), the main focus has always been on projects aimed at restoring the architectural structures [2, 7].

Since the mid-1990s most of the work has concentrated on the reconstruction of the main building in the complex, i.e. the New Castle, which now houses a modernly arranged exhibition on the history of the park and its creator, an information centre, a café as well as seminar and office rooms. From smaller buildings, the orangery (1993) and the utility buildings on the castle grange, where tourist service functions and garden utility buildings are located (2012), have been restored. In some of them the historic function has been restored, whereas the other buildings now house exhibition rooms and catering establishments. The renovation of the Bachelor Hose has begun. Also projects aimed at restoring the smaller architectural structures are steadily being carried out. Moreover, the road surfaces are being restored, the current maintenance of the tree stand and the water system is being conducted, the spatial arrangement is being corrected and the plant system is being shaped. Steadily the decorative elements – flower stands,flower beds and fittings, are being restored.

The Polish part of the park taken over, on behalf of the Ministry of Culture and Art, from the State Forests National Forest Holding, by the Board for the Protection and Conservation of Palace-Garden Complexes, later the Centre for the Protection of Historic Landscape, is currently administered by the National Heritage Board of Poland. The specificity of the initial situation determines a completely different set of problems to be dealt with [8].

The main area on which the conservation efforts concentrate is the reconstruction of the spatial structure obscured in the post-war periodas a result of the discontinuation of maintenance and forest husbandry. The recovery of the park interiors and their characteristic elements, the reconstruction of the viewing interconnections with the west side, necessitate practically continuous sequential work.

The unveiling of Pückler's Stone in 1991, which restored the historic panorama from the east side onto the west side, was from today's perspective a relatively simple action, whereas it took a few years to recover the view from the Mausoleum Terrace in the direction of the New Castle because of the kind and range of the work involved. The next unveilings revealed the next stages, including the clearing of 50 hectares of the 50-year old alder stand obscuring the view, and the drainage of the Reeds Meadow.

The sequential work entails the maintenance of the ancient forest and the further shaping of the plant structure – the regeneration of the system, making up the losses in order to bring back the composition as close as possible to the original design, which is also a condition for preserving its value for the next generations.

An enormous range of work is involved in the reconstruction of the park roads. They have generally survived in the form of road base and accompanying elements, such as stone gutters, limiters, or stones which indicate crossroads. But the missing parts need to be replaced and the surface reconstructed. Owing to the use of the same materials and work methods in the two parts of the Park the reconstructed roads have become an element which strongly integrates the composition on both sides of the Lusatian Neisse river.

A major question is what procedure should be adopted in the case of the structures which have not survived, such as the mausoleum commemorated with a granite cross. The reconstruction of the unpreserved grave – called Stranger's grave is considered. In 2011 reconstruction work in the park retreat called the English House was begun. The road network there has been restored, the foundations of the unpreserved main building have been exposed and one of its pavilions has been restored.

5. CONSERVATION RENOVATIONS OF BRIDGES

So far the conservation of the bridge structures on the park grounds has covered: small bridges in the German part of the park, Prince's Bridge, also called the Royal Bridge (subjected to conservation as the first in 1999) and the Arcade Bridge (its conservation renovation was carried out in 2011) in the Polish part and the reconstruction of the two bridges across the Lusatian Neisse river – the Double Bridge (2003/2004) and the English Bridge (2011) was carried out jointly by the Polish and German administrators. Currently, the third bridge on the Polish side is to be renovated.

5.1. Royal Bridge

Fig. 2. Royal bridge (before 1999)

Fig. 3. Royal bridge (2011)

Originally the Royal (Prince's) Bridge was a wooden structure lined with Virginia creeper. In 1854 in its place the current masonry bridge was erected (Fig. 2). The bridge was renovated in 1999 (Fig. 3). As part of this renovation the whole surface of the bridge was cleaned, the cracked structural masonry elements were rebuilt, necessary damp-proofing work was carried out, a new bridge deck surface was made and the balustrades were reconstructed.

5.2. English Bridge

After many years of efforts the second, after the Double Bridge, bridge on the Lusatian Neisse river was finally built. Preserved historical pillars, the iconography and relics of the balustrade, providing the basis for the restoration of the English Bridge in the form from before the World War II (Fig. 4).

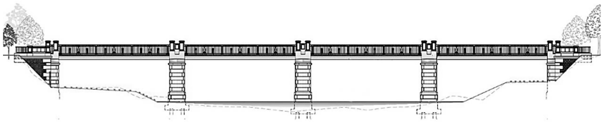

Fig. 4. English Bridge – fragment of reconstruction design (Architekturbüro Bonk + Herrmann, Dresden)

The range of work included dismantling the historic piers, strengthening the foundations and the abutments, sealing the abutments, rebuilding the piers (in the form of a concrete core with granite and brick masonry cladding) using the historic material, repairing the external surfaces of the abutments, making an abutment crown for the bridge steel frame structure, assembling the steel frame structure, the sandstone parts, the pedestrian parapets and the wooden bridge deck.

Fig. 5. English Bridge (ca. 1940) *Fig. 6. English Bridge (ca. 2005)*

Fig. 7. English Bridge (2011)

Fig. 8. English Bridge (2011)

The English Bridge (Figs. 5-8) together with the Double Bridge enable one to drive along the roundabout, patterned on the English "drive" layout, road through the park, whereby one can delight in this great composition and its unique elements. The bridge constituted Pückler's favourite direct connection to the English Cottage – a nonexistent today cottage-style layout with a licensed establishment and an entertainment complex, with a place for dancing, games and a shooting range. This exceptionally lively part of the Park gave name to the English Bridge, and through the association with England and the models from England, was given the name England. Besides the practical function the English Bridge has also a great artistic value and forms a unique architectural point seen from different sides of the Park.

The reconstruction of the two bridges on the Lusatian Neisse beyond the functional aspects of conservation and also has value for the perception of the park on both sides of the border as an integral whole.

5.3. Arcade Bridge

The Arcade Bridge (Figs 9-12) located in the park's east part is one of the three architectural structures which have survived on the Polish side. It is a monumental structure supported by arcades, connecting the edges of a deep tree-covered gorge with Sara's path running on its bottom. The structure was erected in 1853. For the building of the bridge, being at the same a viaduct, besides brick, also glass slag, a byproduct produced in the nearby ironworks, was used. Formed into blocks it adds interesting variety to the structure. The maximum height of this five-span bridge amounts to about 9 m, the width to about 4.5 m and the length to about 28.5 m.

After 1945 wooden beams were introduced into the bridge flooring, to which a temporary wooden balustrade, replacing the destroyed steel balustrade, was secured. The space between the vault, the head walls and the bridge flooring was filled with soil, additionally loading the vaults. The masonry structure was protected by about 25 cm thick layer of cohesive soil and the remaining space was filled with permeable soil in which a system of drains was installed, carrying off rain water seeping through the bridge flooring made up of breakstone with varied fractions ensuring both good compaction and permeability to rain water. For bridges built before the 20th century clay soil would provide insulation. Water is carried out by protruding ceramic pipes.

Prince Pückler did not give this bridge a name. In the plans from 1856 and 1865 the name Viaduct is used. In the plan from 1888 it is referred to as the Viaduct over Sara's Path. In the plan from 1926 the name the Gorge Bridge is mentioned. The currently used vernacular name is the Arcade Bridge. Despite the considerable damage to the balustrade and the weakening of the structure, the bridge was used until 2007, and subsequently was put out of service until 2011 when the renovation work started.

Fig. 9. Arcade Bridge (beginning of 20th c.)

Fig. 10. Arcade Bridge (1990s)

Fig. 11. Arcade Bridge during renovation work (2011)

Fig. 12. Arcade Bridge (2011)

According to the renovation plan, the historic function of the bridge was to be preserved. The bridge was to remain a park structure to be used only by pedestrians and by cars with a weight below 10 t, directly connected with the current operation and maintenance of the park.

For about 150 years the bridge had been subjected to changing weather conditions, i.e. repeated changes in temperature, humidity, etc. Moreover, considering the considerable neglect in the maintenance of the bridge, it can be concluded that the main factor reducing the durability of the Arcade

Bridge was the corrosion of its structural components: the bricks and the glass slag blocks, the washing out of the mortar joining them together and the growing of tree roots into the structural components. Cracks and fractures in the vaults and in the west head wall, the weakening of the piers and damage to structural components due to material loss were found. As a result of the damage the load capacity of the bridge significantly decreased and the structure was found to be locally overloaded. Since the further use of the bridge could result in a structural failure, the bridge had to be put out of service and immediately subjected to renovation.

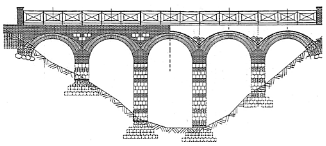

Fig. 13. Arcade Bridge – renovation design [4]

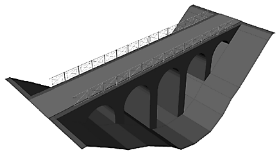

Fig. 14. Arcade Bridge – final spatial model subjected to numerical analysis

As part of renovation work (Figs. 11, 13, 14), the west head wall, pier no. 3 on the east side, the central parts of the vaults, etc., were subjected to crack and fracture injection. Injections increase the strength of the component materials and their load capacity, seal walls and prevent harmful elements, especially moisture and aggressive gases, from penetrating into the walls. Regeneration of the vault and pier systems of a masonry bridge through injections is a very delicate treatment. Currently one can use three methods of strengthening curved masonry structures: gravity injection, pressure injection and vacuum injection.

During the renovation the selected parts of the bridge structure were rebuilt and the missing parts were replaced. This method was applied to both the piers and the brick vaults in order to restore the original geometry of the bridge components. Prior to this it is essential to eliminate the causes of the cracking or fracturing. There are differing views on what binding material should be used. Tests clearly showed that mortars based on cementitious binders, characterized by a much higher strength than the mortars existing on the original elements, had been used for the repairs. The use of mortars with a higher strength than that of the original ceramic material is for many reasons improper and may result in damage. Mortars should be carefully matched to the particular service conditions and trass lime mortars should not be replaced with burned lime or hydrated lime mortars.

An important way of strengthening the brick arches of bridges is through the use of steel tie-back anchors and additional elements bracing the masonry system, which help to reduce wall displacements and coat and rib cracking and may take part in carrying additional tensile forces appearing in the case of damage.

The proper functioning of a steel tieback anchor in historic structures depends on the application and maintenance of a proper force in it. The introduction of a proper tensioning force should produce such a state of stress in the masonry, which will effectively counter the action of the external forces and increase the overall stiffness of the structure. Improper transfer of the force from the tieback to the wall may cause further cracking. In the case of traditional stays, the anchoring of a bearing element (e.g. a metal plate, rosette, flat bar or rod) is usually visible on the façade. Sometimes it is hidden under a masking layer: plaster, rebuilding or a façade ornamental element. This kind of masking was used also in the case of the Arcade Bridge.

The renovation of the brickwork part of the bridge consisted in removing damaged and rotten bricks in the existing parts, and brickwork joints to a depth of at least 20 mm on the whole brickwork surface. The existing brick was cleaned using CP methods (Fig. 15). Then the joints were blown with compressed air and a water jet. The so-called water lance not only cleans, but also wets the elements, which is recommended prior to pointing and filling cavities. The bricks used for the rebuilding were fabricated to order, in imitation of the historic bricks, in the nearby brickyards. The mortars used for masonry work and the ones used for pointing were based on trass binders. The appearance of the masonry after cleaning, luting, replacing the missing bricks and glass blocks and repointing (Fig. 16) recalls the former splendour of this bridge structure. The whole of the brickwork structure and the recovered and renovated stone cornices were protected with hydrophobic preparations. The bridge deck surface layer was made from granitebreakstone, similarly as all the roads.

Fig. 15. Surface of bridge before cleaning

Fig. 16. Surface of bridge after cleaning

The so-called glass slag (Figs. 17, 18) which could be used to replace the damaged blocks is now unavailable since it was a byproduct from the nearby ironworks. Some of this material was obtained through the project German partner – the Fürst-Pückler-Park Bad Muskau Foundation. The other missing elements were made from glazed ceramic blocks coloured in imitation of the glass blocks.

Fig. 17. Arcade Bridge – built-in original structural elements made of glass slag

Fig. 18. Arcade Bridge – original and new elements made of glass slag

The renovation also included damp-proofing the vaults on the extrados side, and the head walls from the inside as well as vertical and horizontal damp-proofing of the piers. Drains carrying rain water off the bridge and a road surface were laid and the cornices and the parapets were reconstructed.

The Arcade Bridge after the conservation renovation was opened on 9 December 2011. The renovation was co-funded from the European Regional Development Fund within the framework of Poland-Saxony Cross-border Cooperation Operational Programme. The main aim of this Operational Programme is to support the sustainable development of the Polish-Saxony support area in order to strengthen its economic and social cohesion.

6. CONCLUSIONS

The Muskau Park (Muskauer Park) is the largest landscaped-style park and one of the finest examples of the 19th century European garden art. Since the end of the 1980s it has been the subject of the Polish-German restoration project. Since 1991 being restored values of the historic composition.

The structural conservation of the bridges is a vital element of this restoration process. Especially that it is probably the only example in Europe of close cooperation of two countries in the protection and conservation of historic objects and the cultural landscape.

The materials (expert opinions, photographs, designs and inventories) relating to the Muskau Park objects, used in this paper come from the resources of National Heritage Board of Poland, the Prince Pückler Park Foundation in Bad Muskau (Stiftung Fürst-Pückler-Park Bad Muskau) and the authors.

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Abstract

The Muskau Park is one of the most extensive 19th century landscape-style park layouts in Europe. From among the numerous landscape-style park compositions created in that period the Muskau Park stands out not only owing to its artistic values, but also because of the fact that the Park, as the consequence of World War II divided by the state border on the Lusatian Neisse river, since the 1980s has been undergoing restoration carried out by two countries: Poland and Germany, within the framework of a joint project. It is owing to the efforts of the Polish side and the German side that this park in 2004 was inscribed on the UNESCO World Heritage List and at the same time became a symbol of the reconciliation between the nations of Poland and Germany and of the conservation collaboration in the field of cultural landscape protection. Thanks to cross-border cooperation measures were taken to restore the Park's historic compositions, including bridges to their former glory. The English Bridge and the Double Bridge, connecting the two banks of the Lusatian Neisse, and the Arcade Bridge situated on the Polish side of the Park have been subjected to conservation renovations whereby the bridges have been restored to their former glory.

Streszczenie

Park Mużakowski jest jednym z najrozleglejszych w Europie XIX wiecznych założeń parkowych w stylu krajobrazowym. Spośród licznych kompozycji w tym stylu powstałych w epoce obiekt ten wyróżniają nie tylko wartości artystyczne, ale i fakt, że park, podzielony w konsekwencji II wojny światowej granicą państwową na Nysie Łużyckiej, jest od końca lat 80 XX wieku rewaloryzowany przez dwa państwa – Polskę i Niemcy w ramach wspólnego projektu. To dzięki staraniom strony polskiej i niemieckiej, obiekt ten został wpisany w 2004 roku na Listę Światowego Dziedzictwa UNESCO, stając się jednocześnie symbolem pojednania między narodami Polski i Niemiec i współpracy konserwatorskiej w dziedzinie ochrony krajobrazu kulturowego. Dzięki współpracy transgranicznej podjęto działania mające na celu przywrócenie świetności parkowym obiektom mostowym. Mosty angielski i podwójny łączące oba brzegi Nysy Łużyckiej jak i most arkadowy znajdujący się po polskiej stronie parku poddano remontom konserwatorskim przywracając im dawną świetność.

Clara Bertolini-Cestari[1], Stefano Invernizzi[2], Antonia Spanò[3], Lara Mallamaci[4]

Laser modeling and Structural assessment of a XVII[th] century wooden dome

Modelowanie laserowe i ocena konstrukcyjna XVII-wiecznej drewnianej kopuły

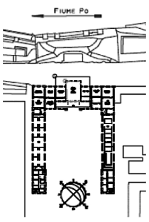
Keywords: Cultural heritage, Wood dome, Structural assessment, Digital photogrammetry, Laser surveying, Orthophotography

Słowa kluczowe: dziedzictwo kulturowe, drewniana kopuła, ocena konstrukcyjna, cyfrowa fotogrametria, pomiar laserowy, ortofotografia

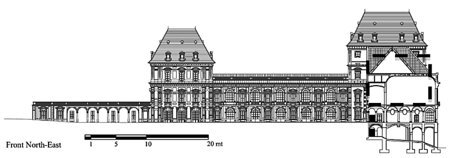
1. INTRODUCTION

The Valentino Castle is placed on the north side of the river Po, and is nowadays fully included in the city of Turin, Italy. The Castle has a very ancient origin, though the first official reference to it dates only 1543 [1]. The Duke of Savoy acquired the asset composed of a palace with garden and, starting from 1620, Christine of France charged the architect Carlo di Castellamonte with some main extension works, including the doubling of the central body and the realization of the towers. In the nineteenth Century the destination of use of the Castle changed several times (Veterinary school, barrack, Royal School of Application for Engineers), and the structure of the Castle was each time modified consequently.

Nowadays, the Castle hosts the Faculty of Architecture of the Politecnico di Torino. The U shaped plan of the Castle is covered with a valuable roof in Vallone's Dark Stone tiles, supported by wooden tables connected to an elaborated wooden frame [2]. A scheme of the plan and longitudinal section of the Castle is shown in Fig. 1.



(a)



(b)

Fig. 1. Plan (a) and longitudinal section (b) of the Castle

The Hall of Honor, located at the first floor of the main body, is now the aula magna of the faculty of Architecture. The Hall has a rectangular shape 16 m long and 11 m wide. The Hall is covered by a pavilion vault, and is fully decorated with frescoes and stuccos.

The vault has been damaged by water infiltration in the past, especially during the last world conflict, due to the fact that the roof was largely damaged. For this reason, the fresco in the central region of the vault is interrupted, and replaced in the past with uniformly colored stucco.

The dome of the Hall is a so-called "fake vault" or "camorcanna" realized with plaster applied on reed mats, which are hanging on a rib wood frame. The typology of the vault can be referred to the Philibert Delorme technique [3].

The principal structure is the above wood rib frame (Fig. 2a) that rules the curved shape of the dome. Each rib is obtained joining together two or more shaped planks with steel nails. The wood planks are 3-6 cm thick, 20-40 cm wide, and usually 2-3 m long. The rib spacing is around 0.7 m. The ribs are connected with steel nails to the underlying orthogonal wood joists. Joist section is 5 cm wide and 2 cm height. The joist spacing is around 25 cm. The continuous reed mat is hanging on the joists, and realizes the surface for the subsequent layers of plaster and stucco for the frescoes.

Since the dome suffered from degradation during the years, a series of interventions were put into place. Among the most relevant, at the end of the XIX[th] century

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A. d'Andrade provided to fill the space between the joists with additional plaster, reinforced with steel nets, connected to the ribs by copper wires. The aim was to contrast the detachment of the vault from the rib structure. Unfortunately, the majority of those interventions added weights to the original structure.

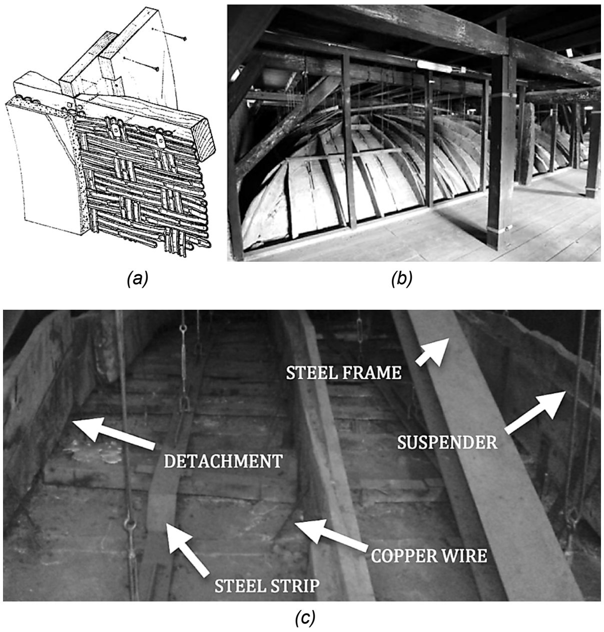


Fig. 2. Scheme of "camorcanna" layers [4] (a), extrados of the Hall of Honor (b) of the Valentino Castle

More recently, in occasion of the celebrations of the first Century of the Italian Republic, the roof structure was renewed and a steel frame structure was placed above the dome, and connected, with vertical suspender cables, to iron strips that were screwed to the little wood carpentry. An overview of the extrados of the dome, with the steel frame structure and the suspenders is shown in Fig. 2b, while some details are shown in Fig. 2c.

Nowadays, a survey revealed that the suspender cables above the vault in the region close to the abutments have lost their tension. This may indicate an increase of the vault deformation; therefore a structural assessment of the dome is mandatory.

The present paper illustrates the results of the structural survey and of the numerical simulation of the dome. In addition, some non-destructive acquisitions are described, which were carried out to support the structural analysis. Finally, possible intervention techniques are discussed, accounting for the restoration criteria of reversibility, minimum intervention, sustainability and compatibility.

2. LASER SURVEY AND MODEL GENERATION

Laser scanning techniques, which enable the generation of 3D models, even with high details level, can properly provide the metric survey of non-planar architectural structures and elements, featured by high complexity of general composition and decoration [5].

Despite a large number of commercial systems are equipped by digital cameras, coaxial with laser distance ray, when a high photographic resolution and precision is needed, integrated systems combining digital photogrammetry and laser clouds acquisition are preferred. [6]

The harmonization and the perfect matching of laser survey and connected models with photogrammetical products are ensured by a set of control points; laser clouds alignment and photogrammetical processes in fact are based on these points that are measured by high precision topographical methods. The topographical networks ensure the single reference system for each surveyed portion of the building, and the control of random blunders diffusion (Fig. 3). In cultural heritage documentation, traditional or innovative 3D types, the main reference system can be local or global; in the study case of Valentino castle, some topographical networks just fixed during other metric documentation activities ensure that every survey updating can be related to the main coordinate systems, which is connected to the regional and urban technical maps (UTM WGS84 reference system).

The vault of the Hall of Honor is featured by a clearly knowable revolution surface, constituted by a cylinder surface based on a polycentric translating arc; there is a rich frescos decoration but not plastic ornamental elements, such as in other lounges, so the laser survey acquisition performed by LEICA HDS6100 phase based scanner has been fulfilled easily with the only encumbrance of the huge chandelier (Fig. 3). 360° points clouds have been acquired in three different position just to front the chandelier shadow cone; they have been registered by control points signalized by reflective target, and results are featured by sub-centimetrical accuracy.

After the complete cloud clearing (clearing of furniture and filtering cloud in order to remove noise) we obtained the points model describing the only interesting surfaces (floor, walls and vault). A large number of revealing section planes have been detected with the aim of analyzing the curves trends

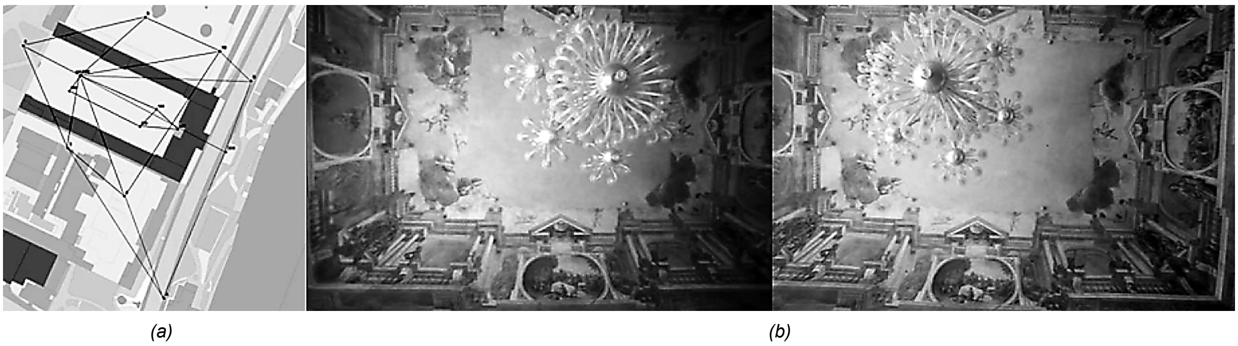
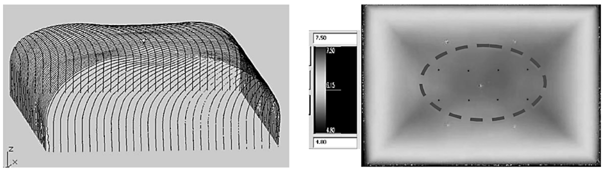


Fig. 3. The first order GPS network fixed for the Valentino castle survey (a), Stereoscopic photograms showing the encumbrance of the chandelier (b)

and spotting the anomalies (horizontal and vertical planes, containing straight lines or generatrix polycentric arcs). After a model shape optimization, several sets of the vaults profiles have been extracted using the commercial tool 3D Reshaper- by Technodigit (Fig. 4a).

The digital elevation model (DEM – Fig. 4b), in which radiometric code of each pixel represent the distance from a plane, in this case horizontal, have been obtained by a cloud decimation until 5% of raw data (from 20 to 100 million points). This surface representation reveals a probable loss of the original geometrical shape: we can estimate a subsiding area near to the central portion of the vault (red dots oval in Fig. 4b). The loggia vault, located close to the main lounge, presents some more precise geometries; the second DEM, derived from another laser model similar to the one we are discussing [7], shows different clarity of intersection bends; we need to consider that such structure has been under restoration during 1980s. The Dem is also used for the correct photogrammetical projection of images on a plane, in order to obtain orthophotos or textured models [8].



(a) (b)

Fig. 4. A laser profile model for visual control of subsidence (a); The Hall of Honor DEM: the red oval identify the subsidence zone, caps shaped, while the light points are fire detectors and the blue points are the computed locations of photograms projection centers (b)

2.1. Digital photogrammetry and orthophotography

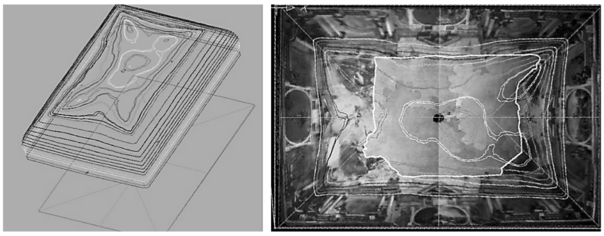
The photogrammetrical survey has been planned in order to generate an orthophoto of the dome [8]. The non metric camera (Canon EOS 1Ds mark II, sensor CMOS 24 × 36 mm 4992 × 3328 pixels) with the calibrated objective (focal length 20.42 mm) has acquired photograms from the floor; since the max shot distance is about 9 m, the embracement is about 16.2 × 10.8 m, i.e. close to the Hall size.

The presence of the chandelier required the acquisition of redundant data. Therefore, 8 photograms, set in two stripes with a high overlapping percentage (80%), have been collected with the pixel size equal to 3 mm (Ground Sampling Distance – GSD). The orientation solution, performed by a single block processing (bundle block method) has provided very good results, with round minus square on measured points equal to few millimeters.

The analysis of section profiles, set in the horizontal or vertical planes and compared with the hypothetical original bending arcs, has enabled the chance to ascertain several localized deformation of the vault, in the order of few centimeters. The surface contours of the vault, computed with a equidistance of 5 cm, presents an high level of geometrical aberration (Fig. 6a), that must be considered in the structural assessment of the vault.

The location of height anomalies obtained from the laser model, and the shape of the frescos abruption outlined in the orthophoto, are in good agreement when results are reported to the same reference system (Fig. 6b).

The asymmetrical trends of contours seems to be connected with the ambit of the vault restoration, and the careful examination of discards measuring the distance in selected points of the actual profiles from the expected ones.

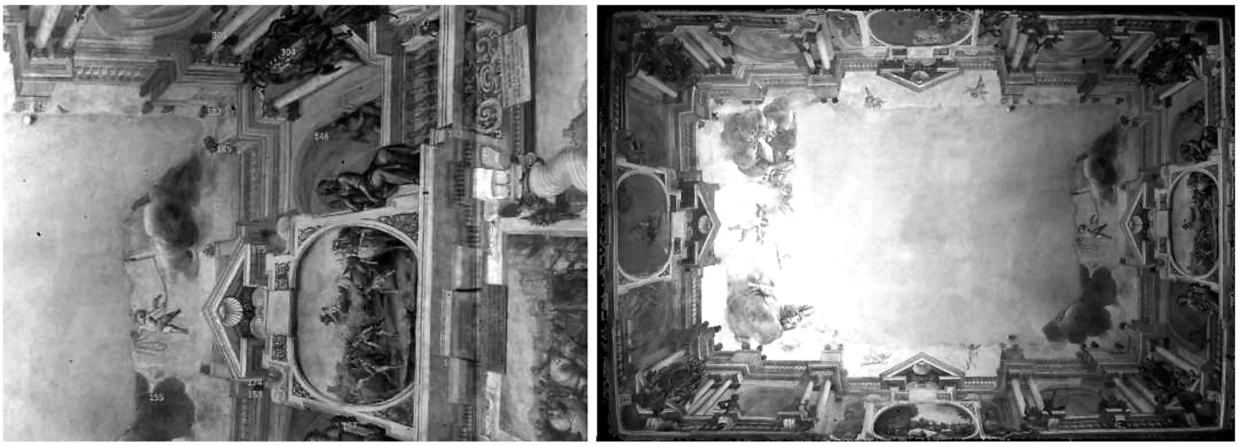


(a) (b)

Fig. 6. Superimposition of the referenced orthophoto with the section profiles (a); final orthophoto of the dome, obtained by a mosaic of projected photograms (b)

3. MATERIALS CHARACTERIZATION

For historic structures, quality assessments of members allow for the maximum retention of original material. The preservation of original structural fabric and associated construction conserves both the cultural significance of the building including architectural qualities and building techniques and the historic and socially important aspects associated with the structure. Furthermore, gaining additional understanding of building material durability, capacity, behavior and



(a) (b)

Fig. 5 Control points for the computing of photogrammetric solution are natural points on the frescos (a); final orthophoto of the dome, obtained by a mosaic of projected photograms (b)

(a) (b) (c)

Fig. 7. Sampling of the wooden structure for identification: from the joist (a); from the rib (b); from the reed mat (c)

use, as well as building techniques and craftsmanship from existing structures provides knowledge that can be applied to present-day construction. A quality assessment begins with the assessment of the members and components that make up the structure as a whole. Samples for the identification of species concerning the main structural elements of the vault are shown in Fig. 7.

The wood ribs of the vault are made of poplar (*Populus sp.*). In order to assess the quality of the wood, a semi-destructive campaign has been performed with resistographic drilling.

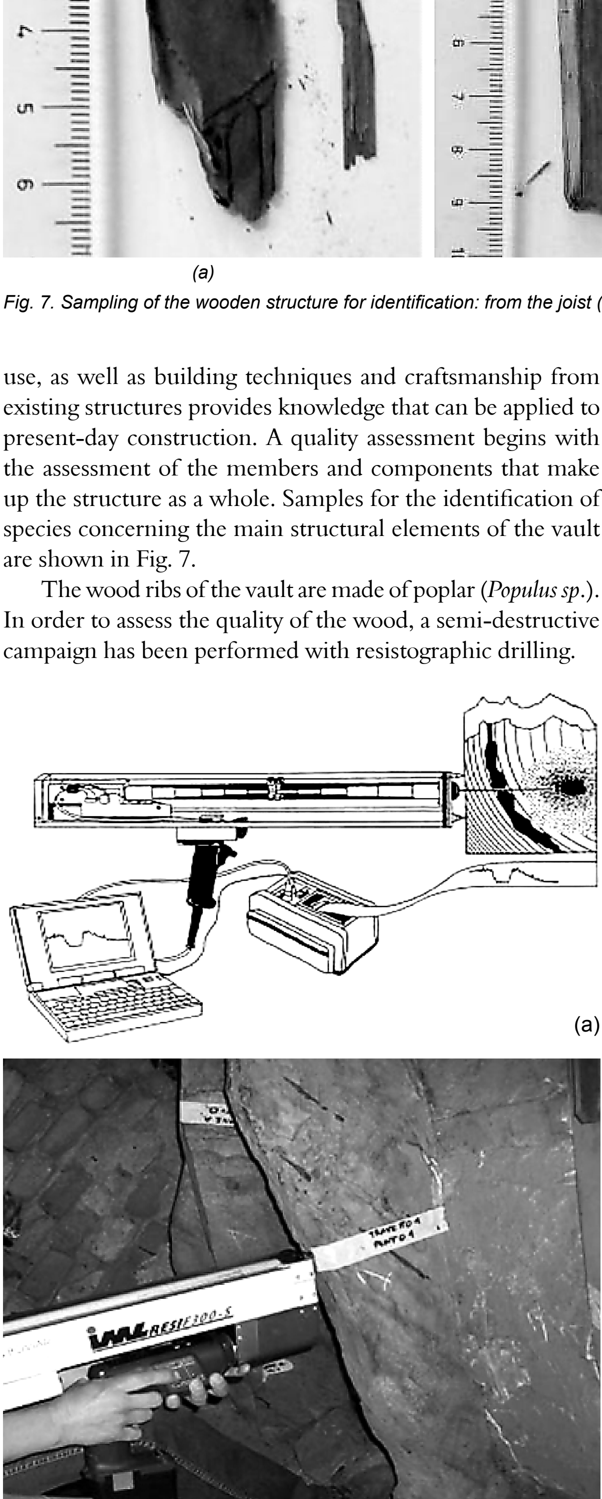
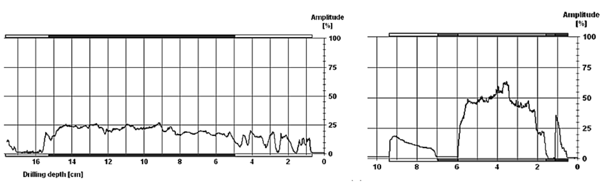


Fig. 8. Sampling of the wooden structure for identification: from the joist (a); from the rib (b); and from the reed mat (c)

Resistance drilling offers a non-destructive means of analyzing the quality of the interior material in wood members. Resistance drills use small diameter (1.5-3.0 mm [0.6-0.12 in]) needle-like drills to bore into timber members and measures the resistance the drill bit encounters as a function of the penetrated depth. Resistance drills have electric motors and are battery operated, offering portability for field investigations. Drill bits are flexible, tungsten steel-tipped needles that will vary in length depending on manufacturer. The needle has to be replaced after 50 to 100 drillings, depending on manufacturer and use.

The drill bit is advanced and rotated at a constant speed throughout the drilling. The torque required to maintain the constant cutting speed corresponds to resistance and is recorded and graphed with respect to the penetration depth. Graphing of resistance data can be done with paper strips, wax paper, or recorded and stored electronically on computer (Fig. 7). Peaks in drilling plots correspond to higher resistance or density, while dips and low points are associated with lower resistance and density.

The resistographic drilling sampling was performed on some of the ribs of the vault, drilling in the two directions perpendicular to the rib longitudinal axis, respectively along the rib height and along the rib thickness (Fig. 9). When drilling is performed along the thickness (Fig. 9b) of a rib composed by three planks, the two discontinuities are clearly recognizable.



(a) (b)

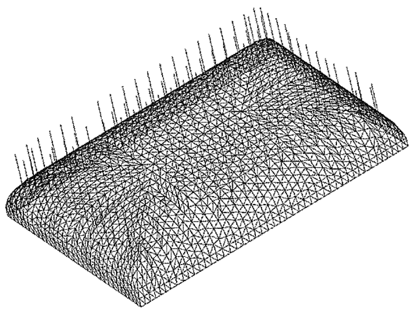
Fig. 9 Resistographic drilling diagrams: along the rib height (a); along the rib thickness (b)

The amplitude in the resistograph diagrams reveals a wood quality ranging from good to very good. It is worth noting that such values are hardly ever encountered when testing nowadays poplar samples. In fact, historical poplar, grown without intense cultivation program, and in a quite colder environment with respect to nowadays, is a much tougher material.

On the contrary, the efficiency of the connections is very hard to assess, although each rib is composed by two or more planks, and head joints are not overlapping.

4. PRELIMINARY FINITE ELEMENT MODEL

A preliminary Finite Element model has been set up in order to understand the structural behaviour of the vault. The model accounts for the exact geometry of the vault, according to the laser survey and the corresponding initial geometry extrapolation. In addition, all the main structural components are considered: the shell and the rib frame, as well as the iron strips and the suspenders. The shell represents the behaviour of the plaster, of the reed mat and of the wood joists in a single equivalent layer. The linear analysis has been carried out with the finite element program DIANA [9]. The overall mash of the model is shown in Fig. 10a, while the table in Fig. 10b reports the basic mechanical assumption.



(a)

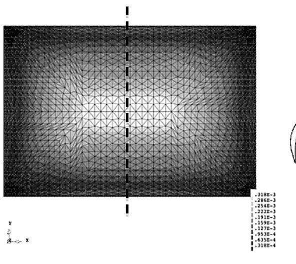
	E [Pa]	ν	ρ [kg/m³]	type
Suspenders	2.1E+11	0.3	7850	truss
Ribs	8E+9	0.2	510	beam
Strips	2.1E+11	0.3	7850	beam
Vault	13E+9	0.3	1500	shell

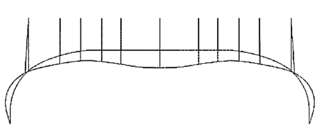
(b)

Fig. 10. Mesh of the model (a); mechanical parameters of the model (b)

The model is subjected to the only action of dead load due to gravity. At the present stage, the nonlinear behaviour of the materials has not been considered yet. Therefore, the model can provide only preliminary information about the structural behaviour of the dome.

Nevertheless, the obtained deformed shape, shown in Fig. 11, is in agreement with the anomalies measured by the laser-scannig survey. In particular, the annular region around the big chandelier appears to be the most prone to displacements. On the other side, the suspenders, which are located in the outer region of the vault, are not elongated at all. This corresponds well with the evidence of the survey, which reveals that many of them have actually lost their tension.





(a) *(b)*

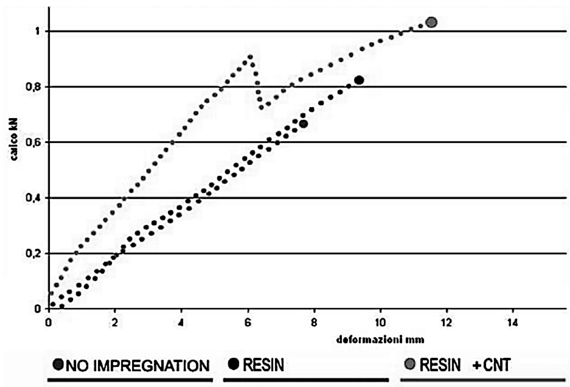
Fig. 11. Contour plot of vertical displacements (a); transverse section with vertical displacements (b)

5. CONCLUSIONS

The case study of the Hall of Honor of the Valentino Castle in Torino, Itlay, is presented. The detailed laser survey, and ortophotography allowed for an accurate modelization of the vault geometry at the present state, and the localization of the main geometrical anomalies.

This information is combined with some non-destructive analyses and with the structural survey of the extrados of the vault. The preliminary finite element model is confirmed as far as the deformed shape is concerned, and also the loosen suspenders can be localized.

(a)



(b)

Fig. 12. Bending tests and scheme for the mechanical resistance on timber "Jupiter" joints (a); load displacement diagram: positive effect of the reinforcement (b)

The extraordinary maintenance of the vault has to be carried out adopting very low invasive, but very innovative techniques. This is the subject of the project M.A.N. for the consolidation and reinforcement of historical wood structures with nanoreinforced composites. Such techniques have been already tested on historical samples, characterized by the classical "Jupiter" joint connection [10]. The first results obtained from the laboratory tests are quite promising, as can be see by Fig. 12b; therefore the technique will be applied in the described case study.

ACKNOWLEDGEMENTS

The financial support provided by the Piedmont Region (Italy) to the POR-FESR Project "Nanomateriali per la Manutenzione ed i Recupero di Manufatti Lignei" (M.A.N.) is gratefully acknowledged.

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Abstract

The main room of the Valentino Castle in Torino, the "Salone delle Feste", was conceived and realised in the XVIIth century under the guidance of Carlo di Castellamonte. The beautiful frescos and stuccoes of the domical vault are sustained by a typical Delorne carpentry, whose span is among the largest of their kind. The dome suffered from degradation during the years, and a series of interventions were put into place.

Nowadays, a survey revealed that the suspender cables above the vault in the region close to the abutments have lost their tension. This may indicate an increase of the vault deformation; therefore a structural assessment of the dome is mandatory.

In order to reinforce the structural hypothesis of damages and to reveal the deformation effects, a high detailed metric survey have been carried out with integrated laser scanning and digital close range photogrammetry. The photogrammetrical survey of frescos, with the re-projection of images on vault surface model (texture mapping), is purposed to exactly localize formers restoration and their signs on frescos continuity.

The present paper illustrates the generation of the 3D high-resolution model and its relations with the results of the structural survey; both of them support the Finite Element numerical simulation of the dome.

Finally, possible intervention techniques are mentioned, accounting for the restoration criteria of reversibility, minimum intervention, sustainability and compatibility.

Streszczenie

Główna sala na zamku Valentino w Turynie, zwana "Salone delle Feste", została zaprojektowana i zrealizowana w XVII wieku przez zespół pod kierunkiem Carlo di Castellamonte. Piękne freski i stiuki na sklepieniu są wykonane na konstrukcji ciesielskiej typu Delorne, której rozpiętość jest jedną z największych pośród konstrukcji tego typu. Przez lata kopuła systematycznie niszczała, w związku z tym podjęto serię prac interwencyjnych.

Obecnie badania ujawniły, że ściągi umieszczone powyżej sklepienia, w pobliżu przypór, straciły napięcie. Może to wskazywać na wzrastającą deformację sklepienia, dlatego niezbędne jest przeprowadzenie oceny konstrukcyjnej kopuły.

W celu potwierdzenia oceny stanu konstrukcji dotyczącej zniszczeń oraz wykrycia skutków deformacji, została przeprowadzona bardzo szczegółowa ekspertyza metryczna przy użyciu zintegrowanego skanowania laserowego i cyfrowej fotogrametrii bliskiego zasięgu. Badania fotogrametryczne fresków, oraz ponowna projekcja obrazów na modelu powierzchni sklepienia (mapowanie tekstury), miały na celu dokładną lokalizację wcześniejszych działań restauratorskich i ich śladów w ciągłości fresków.

Niniejsza praca ilustruje proces tworzenia modelu 3D o wysokiej rozdzielczości, oraz jego odniesienie do rezultatów ekspertyzy konstrukcyjnej, oparte na numerycznej symulacji kopuły metodą elementów skończonych.

Na koniec wymieniono działania, które można podjąć w celu ratowania obiektu, uwzględniające restauratorskie kryteria odwracalności, minimalnej interwencji, trwałości i zgodności z pierwowzorem.

Jacek Grosel[1], Wojciech Sawicki[2], Zbigniew Wójcicki[3]

Vibration measurements in analysis of historical structures

Pomiary i analizy drgań obiektów historycznych

Keywords: Vibrations, Measurement, Analysis, Historical constructions, OMA

Słowa kluczowe: drgania, pomiary, analizy, konstrukcje historyczne, OMA

1. BASIC INFORMATION ON DYNAMIC MEASUREMENTS

1.1. General information

The paper presents contemporary methods of historical structures analyses. These analyses are based on the interrelations among *in situ* dynamic measurements, analysis of the obtained data, and calculation on a FEM model. Thanks to the merging of experimental study with calculation techniques, it becomes possible to perform analyses of valuable historical structures in a way that is safer, more exact and better suited to the real dynamic and static behaviour of the structure. Where the possibilities of theoretical analyses end, experimental studies begin and *vice versa*. Experimental and theoretical models validate each other, which results in a much better customisation of analysis procedures to the studied object and the acquisition of much more precise results, which are simultaneously more reliable and closer to reality. Unfortunately, these studies require the use of very expensive and complex measuring instruments, very expensive and very advanced software and the necessary knowledge and research experience. There are not many scientific research centres in Poland, in Europe or in the world that possess all of the above, but there are some. The Structural Dynamics Division of Civil Engineering Institute of Wrocław University of Technology is one of them.

1.2. System PULSE™ – Brüel & Kjær's platform for vibration analysis

Dynamic measurements were performed with the use of a Brüel & Kjær 34 channel PULSE™ system, (Fig. 1).

The PULSE™ system we use can be divided into two independent parts, each of 17 input channels, all featuring the frequency range of DC to 25.6 kHz and 12 additional auxiliary channels. The separate power supply units enable using 110 – 240 V AC, 10 – 32 V DC, built in batteries, external power packs and portable, petrol driven generators. Being based on type 3560 PULSE system and its Dyn-X acquisition modules, all inputs reach the dynamic range of 160 dB with ideal linearity and phase matching. All of this, together with TEDS technology and the extensive diagnostics of the input chain condition, makes the data acquisition practically unattended, mostly because there is no need to control the input range settings anymore.

Fig. 1. Brüel & Kjær's PULSE system with 34 measure channels

1.2.1. Transducers and accessories for Experimental Modal Analysis (OMA and EMA)

The system has been equipped with transducers (Fig. 2) intended for the purpose of experimental Modal Analysis: a set of 16 seismic accelerometers DeltaTron 8340 (big mass – 775 g, high sensitivity 1000 mV/ms^{-2}) and a set of 32 mini-accelerometers THETASHEAR 4507 8340 (little mass – 4,8 g, lower sensitivity – 100 mV/ms^{-2}), dynamic PCB strain gauge, and a laser vibrometer, equipped with a shooting telescope, (Fig. 2).

In civil engineering, and especially when performing modal analysis, very sensitive accelerometers are needed. Unfortunately, the more sensitive the heavier the accelerometer is – and the more difficult to fit onto elements of the

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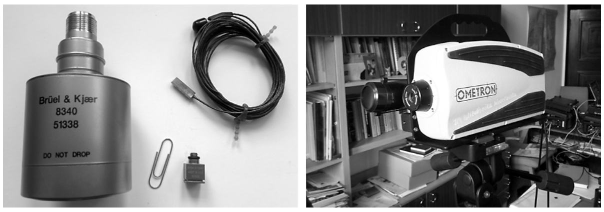

Fig. 2. A seismic accelerometer DeltaTron 8340, mini-accelerometer THETASHEAR 4507, PCB dynamic strain gauge (on the left) and Laser Doppler Vibrometer (on the right)

historical structure. An alternative solution is to use a Laser Doppler Vibrometer, which performs non-contact vibration velocity measurements and can be attached to the PULSE™ system alongside accelerometers and become one of the sensors registering vibration.

1.2.2. Accessories intended for the purpose of Experimental Modal Analysis

Additionally, the system has been equipped with accessories intended for the purpose of Experimental Modal Analysis: a small and large impact hammers (Fig. 3), shakers, calibrators etc.

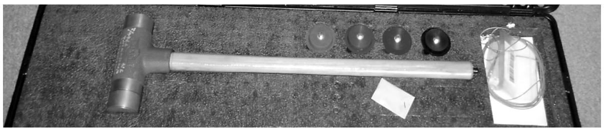

Fig. 3. Large impact hammer (5,448 kg)

1.2.3. Experimental Operational Modal Analysis (OMA)

The most important part of the PULSE™ system is the software, which allows to perform complex analyses of vibration. Special attention shall be paid to the Structural Analysis package, covering all kinds of OMA, Operational Deflection Shapes (ODS) and EMA (shaker, hammer) with structural modifications and simulations. The OMA application has been equipped with all of the newest achievements in that field, including automatic search and detection of mode shapes and also automatic detection and removal of harmonic contents in the measured signal.

The OMA package contains 6 algorithms for obtaining eigenfrequencies and eigenforms:

- FDD (Frequency Domain Decomposition)
- EFDD (Enhanced Frequency Domain Decomposition)
- CFDD (Curve-Fit Frequency Domain Decomposition)
- SSI-UPC (Stochastic Subspace Identification-Unweighted Principle Components)
- SSI-PC (Stochastic Subspace Identification-Principle Components)
- SSI-CVA (Stochastic Subspace Identification-Canonical Variate Analysis)

1.3. Simple dynamic analysis

1.3.1. Time histories

The typical procedure in simple experimental dynamic analysis is to measure and register the time history of accelerations, velocities or displacements. The exemplary registered time histories of the accelerations are shown in (Fig. 4).

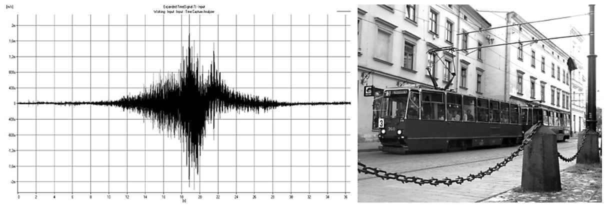

Fig. 4. Exemplary time history of accelerations (passage of a tram, Kraków, Dominikańska Str.)

A measurement taken in order to evaluate the dynamic work of a crack in the Ossolineum library in Wrocław is shown in (Fig. 5).

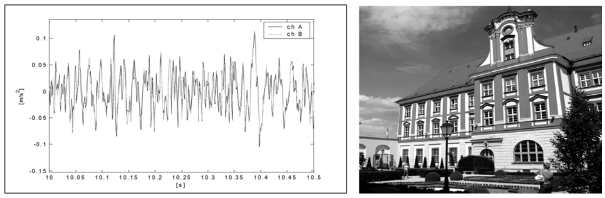

Fig. 5. Exemplary time history of accelerations (on the left) in 2 points on both sides of the crack during the passage of a tram near the Ossolineum library in Wrocław (on the right)

1.3.2. Spectral analysis

The classic spectral analysis (FFT) or Autospectrum (Fig. 6) can be performed as a standard procedure in dynamic analysis. These methods make it is possible to extract dominant frequencies in the signal.

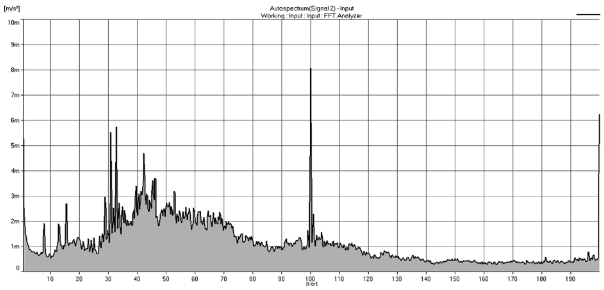

Fig. 6. Exemplary Autospectrum of accelerations (FFT analysis)

1.3.3. Frequency-time spectrogram

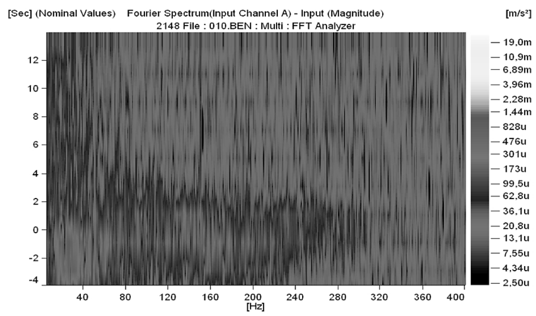

Fig. 7. Exemplary spectrogram of accelerations (frequency-time analysis) of signal shown in (Fig. 4)

The frequency-time analysis (Fig. 7) can also be performed and can be very useful. This analysis makes it possible to observe the changes of the dominant frequencies in time. (Fig. 7) shows a spectrogram of accelerations registered on the wall of the historic building of the Ossolineum library in Wrocław, during the passage of a tram nearby. The figure shows an amplification of the signal, which may be observed with the change of both the time and the frequency domain. The signal is strongest in the frequency range 10-40 Hz.

2. SIMPLIFIED ANALYSIS ACCORDING TO NATIONAL STANDARD

The dynamic loads acting on historical structures are most often due to ground movement. The movement may be caused by traffic (cars, trains, trams, metro) or by seismic or para-seismic excitation. In order to evaluate the structure strain, a full static and dynamic analysis of the structure under a kinematic load may be performed. Such analyses are now performed with the use of FEM. Simplified dynamic analyses are performed by comparing the obtained measurements with given guideline values, which have been determined on the basis of experience of previously observed cases. On a diagram, the placement of the measurement values in reference to these guideline values (above or below the curves of guideline values) indicates how the vibration should be classified. Two instances of dynamic analyses performed according to these guideline values are presented.

2.1. Polish Standard PN-85/B-02170

The Polish standard, entitled *Evaluation of the harmfulness of building vibrations due to ground motion*, allows for the use of simplified analysis in the study of buildings under the following conditions: the buildings must be masonry, with a number of limitations concerning the number of storeys and the dimensions in projection. Two scales have been developed:

- SWD-I for small buildings (max. 2 storeys, the width or length of building less than 15 m).
- SWD-II for larger buildings (up to 5 storeys, whose heights are smaller than the doubled width of the building).

In cases of evaluation performed with the use of the SWD (scale of dynamic influence), the measuring point is placed on the building fundament on the side of the vibration source, or on the bearing wall on the level of the surrounding ground. In this point, two-axis (diaxial) measurements with two horizontal axes are taken.

If the excitation is a harmonic signal, then the excitation frequency and amplitude are determined – the two coordinates of the point whose position on the SWD scale diagrams, compared to the curves for guideline values specified in these diagrams, determines the degree of dynamic harmfulness. However, if the excitation is more complex, an analysis corresponding to the CPB (Constant Percentage Bandwidth) filtering in acoustics should be performed. The registered signal is filtered through 1/3 octave filters. An octave is a range of frequencies whose upper limit is twice the value of its lower limit; a 1/3 octave is one of three parts of the octave, each of which fulfills the condition that the ratio of the upper and lower limits is constant. For each 1/3 octave an amplitude is determined that, together with the middle frequency of the range, yields the coordinates of the point on the SWD diagram. The analysis is performed by placing the above points on a coordinates system, in which the horizontal axis is the axis of frequencies and the vertical one is the axis of acceleration or displacement amplitudes, and determining which zone the points belong to. In the abovementioned coordinates system, four division lines A,B,C,D were determined, dividing the zones of dynamic influence (zone I, II, III, IV, V). „The following criteria of dividing the zone of harmfulness were assumed:

- zone I – vibration imperceptible to the building,
- zone II – vibration perceptible to the building but not harmful to the construction; an accelerated wear of the building, first cracks in the [...] plaster,
- zone III – vibration harmful for the building, causing local cracks and fissures [...],
- zone IV – vibration highly harmful for the building [...],
- zone V – vibration causes construction failure through the collapse of walls and ceilings".

Although it is not explicitly stated in the standard, in our opinion, the level of vibration in historical structures should not exceed the values determined by line A.

The greatest disadvantage of the Polish standard PN-85/B-02170 is that it cannot be applied to large-scale structures (churches, chapels *etc.*) and it does not introduce special vibration limits that should not be exceeded in historical structures. [Fragments of standard translated by the authors].

2.2. German Standard DIN 4150-3

In dynamic analyses of historical structures and monuments, the German norm DIN 4150-3 – *Effect of vibration on structures* is much more useful than its Polish counterpart. The norm unequivocally refers to historical structures ("listed buildings under preservation order"), (Table 1), for which it introduces separate maximum permissible levels of vibration (ones much lower than for other types of structures). Moreover, in contrast to industrial structures, even minor damage to historical structures (together with one other type of structures) is considered to reduce their serviceability. Such minor damage includes the appearance of cracks in the plaster, the enlargement of already existing cracks, and the detachment of partitions from loadbearing walls and floors.

There are no limitations to the structure size in the German standard, so it may be applied to large structures such as churches, chapels etc. The standard introduces two categories of vibration: short-term and long-term. Short-term vibration is "[v]ibration which does not occur often enough to cause structural fatigue and which does not produce resonance in the structure being evaluated," while "long-term" refers to all other types of vibration.

In evaluating the effects of short-term vibration on the structure as a whole, the procedure [2] is:

- "Evaluations as in this standard are based on the maximum absolute value of the velocity signals, $|v|_{i,\max}$, for the three components (where $i = x, y$ or z) of the unweighted velocity signals $v_i(t)$, measured on the building foundation (this parameter is referred to below as *vi* for short)".
- "The vibration measured in the plane of the highest floor resting on external walls also provides significant information for this evaluation, taking the maximum of the two horizontal components as a basis. Measurements taken at that point [...] may be used to determine the horizontal response of the structure to the excitation at the foundation".

(Table 1) and (Fig. 11) "... give guideline values for *vi* at the foundation and in the plane of the highest floor of various types of building. Experience has shown that if these values are compiled with, damage that reduces the serviceability of the building will not occur. If damage nevertheless occurs, it is to be assumed that other causes are responsible. Exceeding the values at Table 1 does not necessarily lead to damage: should they be significantly exceeded, however, investigations are necessary". To determine which frequency range shown in (Table 1) should be applied, the amplitude of the relevant velocity and the corresponding frequency should be taken under consideration.

Moreover, in cases when "short-term vibration causes floor to vibrate, if v_z is no greater than 20 mm/s when measured at the point of maximum velocity (which is usually at the centre of the floor), a reduction in the serviceability of the floor is not to be expected". "To measure vibration in foundations, the pick-ups for the three directions of measurement shall be placed close together on the ground floor of the building to be investigated, either at the foundation of the outer wall, on the outer wall itself, or in a recess in that wall. In buildings without a basement, the point of measurement shall be no more than 0,5 m above the ground. Measurement points shall preferably be on the side of the structure that faces the source of excitation. The time history of the vertical vibration (z-axis) and horizontal vibration (x – and y-axes, at right angles to each other) shall be recorded, with one of the directions of measurement running parallel to a side wall of the building. For structures with a large ground floor area, simultaneous measurements shall be made at several locations".

In evaluating the effects of long-term vibration on the structure as a whole, (Table 1) "... gives guideline values for the highest value of the two horizontal components measured in the top floor, for different types of building." For historical structures, the value 2,5 mm/s (last row, last column of (Table 1)) is the guideline value – the curve of the guideline value is the purple dashed line in (Fig. 11).

Table 1. (Compiled from Table 1 and Table 3 of the standard DIN 4150-3)

Line	Type of structure	Guideline values for velocity, vi, in mm/s				
		Vibration at the foundation at a frequency of			Vibration at horizontal plane of highest floor at all frequencies	
		1Hz to 10Hz	10Hz to 50Hz	50Hz to 100Hz*)	Short-term	Long-term
1	Buildings used for commercial purposes, industrial buildings, and buildings of similar design	20	20 to 40	40 to 50	40	10
2	Dwellings and buildings, of similar design and/or occupancy	5	5 to 15	15 to 20	15	5
3	Structures that, because of their particular sensitivity to vibration, cannot be classified under lines 1 and 2 and are of great intrinsic value (e.g. listed buildings under preservation order)	3	3 to 8	8 to 10	8	2.5
*) At frequencies above 100 Hz, the values given in this column may be used as minimum values.						

The great advantage of the German standard DIN 4150-3 is that it clearly discusses historical structures – curve 3 in (Fig.11). Its disadvantage is the difficulty in interpreting some of the standard's references: in the right column in (Table 1), the standard refers to the vibration of the structure's "highest floor". In the case of churches and other hall (nave) buildings, it is difficult to determine what the term "highest floor" actually means. Similarly, it is somewhat difficult in practice to classify vibration as short – or long-term on the basis of the standard's definition. In such cases, the suitable procedures for short – and long-term vibrations should both be used.

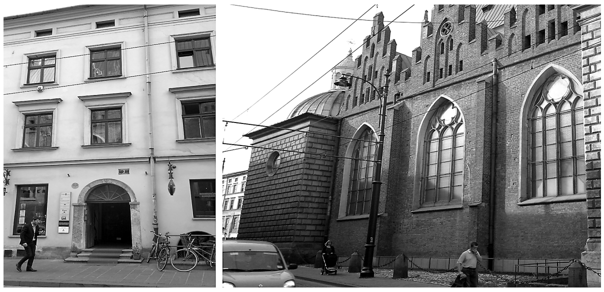

Fig. 8. On the left: No. 3 Dominikańska street. On the right: the Holy Trinity Church from the direction of Dominikańska street. From left to right, the church's chapels are: Lubomirski Family Chapel, St. Thomas' Chapel, the Saviour's Chapel, St. Joseph's Chapel, St. Dominic's Chapel (chapel of the Myszkowski family), and the Chapel of the Rosary

2.3. Example 1 – vibration analysis on Dominikańska street in Kraków

The Holy Trinity Church and the building in Dominikańska street in Kraków are shown in (Fig. 8).

On the side of the church facing the street, along which trams pass, there is a number of chapels. The study of these two buildings is an example of a case where both the Polish [1] and the German [2] standards had to be used: the Polish one for the evaluation of the historical building, and the German – for the evaluation of the church.

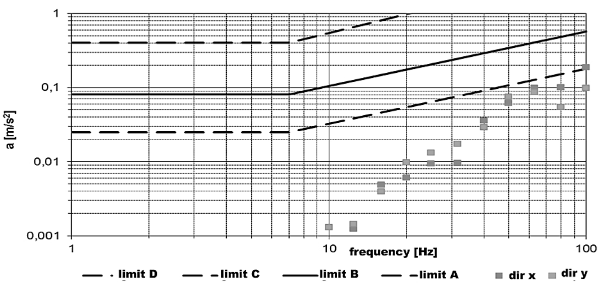

Fig. 9. Evaluation of the harmfulness of ground-transmitted vibration according to the SWD-II scale

In (Fig. 9), the dynamic analysis of vibration of the building on Dominikańska street (Fig. 8, left) due to the passage of a tram, performed according to the Polish standard [1], is presented. It follows from the presented results that the maximum values of acceleration slightly exceed the limit A in one of the 1/3 octaves (the highest one). Limit A is "the lower limit of vibration perceptibility to the building and the lower limit of non-negligible dynamic influence […which causes] vibration perceptible to the building, but not harmful for its structure; only accelerated building wear and the first cracks in […] plaster *etc.* appear." [1]

The dynamic analysis of the vibration of the Holy Trinity Church (Fig. 8, right) due to the passage of a tram was performed according to the German standard. A triaxial measuring system made up of three DeltaTron 8340 accelerometers, fitted to the measuring point on the church wall, is shown in (Fig. 10).

Measurements were taken near the Chapel of the Rosary and the Lubomirski Chapel on the ground floor of the building, i.e. on the level of – 1,4 m below ground. Velocities of vibration were measured in three directions perpendicular to one another, in accordance with [2]. It was found that the amplitudes of these velocities are very small, as shown by the yellow point which, in (Fig. 11), lies well beneath the red curve delineating guideline values for short-term vibration in historical structures. An additional purple dashed line in (Fig. 11) indicates the permissible level of long-term vibration for historical structures (in accordance with (Table 1)). While the passage of one tram is in itself a source of short-term vibration, the traffic on Dominikańska street (which can reach the volume of up to one tram every 0,5 minute in rush hours) may, according to [2], be classified as a source of long-term vibration.

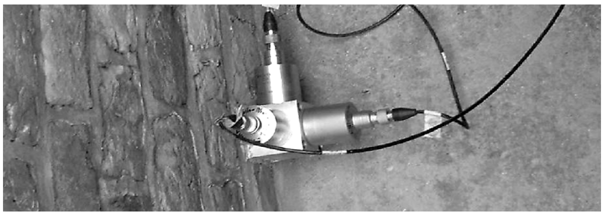

Fig. 10. A triaxial measuring system made up by three DeltaTron 8340 accelerometers

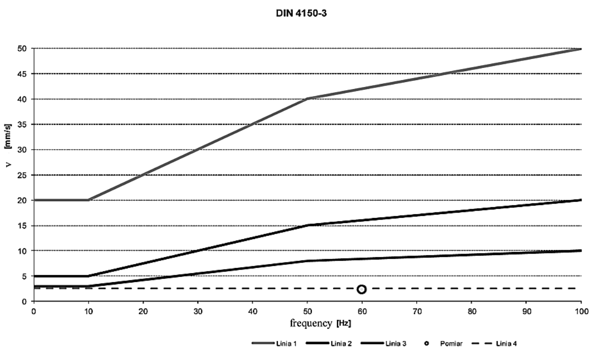

Fig. 11. Vibration influence on the structure of the Holy Trinity Church according to DIN 4150-3

In conclusion, the influence of the vibration due to tram traffic on the structure of the Holy Trinity Church is small. However, in the long term, the appearance of new cracks in the chapel walls is to be expected – as well as the enlargement of already existing cracks, which are even now in evidence as shown in (Fig. 12).

The level of vibration is directly dependent on the velocity of the tram (Fig. 13) that causes it; when the speed of the tram rises from 15 km/h to 40 km/h, the church vibration level doubles.

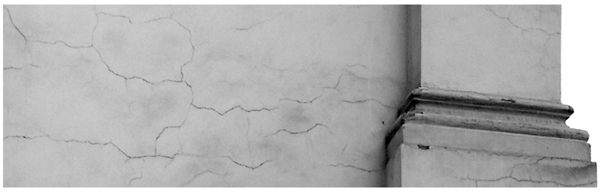

Fig. 12. Cracks in the chapel walls of the Holy Trinity Church

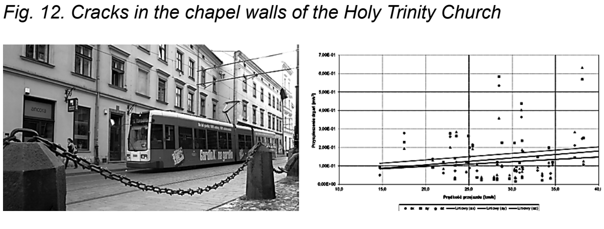

Fig. 13 . "Bombardier NGT6" tram on the Dominikańska street; dependence of vibration level on tram velocity

As a result of the presented study, a recommendation was made to maintain the existing speed limit of 20 km/h for trams passing along the Dominikańska street. As the average recorded speed of trams was 27,8 km/h, and more than 90% of the recorded trams exceeded the speed limit, it was also recommended that the limit be enforced more rigidly in the future.

3. OPERATIONAL MODAL ANALYSIS (OMA)

3.1. The idea of modal analysis

The main idea of modal analysis is to obtain modal parameters H(ω) – modal frequencies (eigenfrequencies), modal forms (eigenforms) and modal damping – on the basis of a measured excitation X(ω) and the structure's response to this excitation Y(ω) (Fig. 14). In civil engineering structures, the greatest difficulty lies in controlling the excitation of vibration while simultaneously measuring the exciting force. For this reason, Experimental Modal Analysis (EMA) is only used in laboratory conditions. In EMA, sufficient force must be applied in order to excite vibration of the studied structure that will be strong enough to be measurable by the sensors. Modal hammers or special vibration exciters are used for this. Both the frequency and the value of the applied force must be known (measured).

In OMA, there is no need to measure the force of excitation, as it is sufficient to measure the system's response to the existing exploitation excitation. However, OMA requires more exact and much longer measurements than EMA. OMA assumes that the excitation – X(ω) – is, theoretically, white noise. Usually, in longer recording time, the real excitation approximately fulfills this assumption (as long as it is not caused by the functioning of machines). Therefore, OMA requires long measurements performed with the use of highly sensitive transducers.

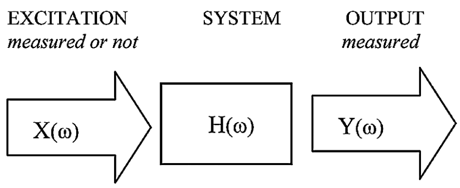

Fig. 14. Measurement-system model for modal analysis

The modal values obtained thanks to OMA may be used to more accurately determine the properties of the model, to add precision to the calculation scheme (which is especially important where it is impossible to take samples for analysis) and to validate the FEM model. OMA may also be used to monitor the structure's technical condition. With any changes in this condition (appearance or enlargement of cracks, material degradation and therefore stiffness change), a change will appear in the modal properties of the structure. Example 2 presents an attempt of using OMA to establish the crack state of the structure.

3.2. Example 2 – Aula Leopoldina in Wrocław

3.2.1. Description of the object and the study

The structure of the ceiling above the Aula Leopoldina is complex, [3, 4]. The main structural elements are wooden deal beams, supported on opposite walls. In the beginning of the twentieth century, the structure was reinforced by introducing

Fig. 15. Main Building of Wrocław University, view from Odra River and the interior of Aula Leopoldina

Fig. 16. View of the Balzer Hall above the Aula Leopoldina. Placement of measuring points in the Balzer Hall is shown in (Fig. 17) and model OMA in (Fig. 18).

additional steel plate girders over every twentieth beam. The planks are supported from below by a c-profile, placed perpendicularly to the beams at about 1/8 of their length from both ends (and thus from the supporting walls). Below this underlying structure, the ceiling of the Aula Leopoldina is covered in valuable baroque paintings (Fig. 15).

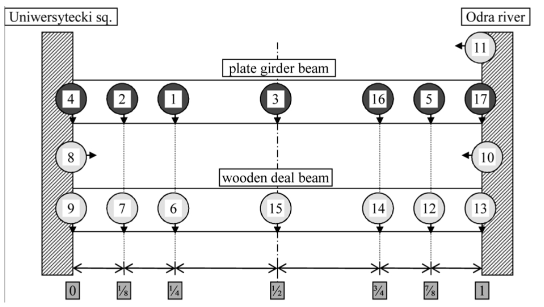

Fig. 17. Diagram of accelerometer placement: THETASHEAR 4507– red colour, DeltaTron 8340 – yellow colour

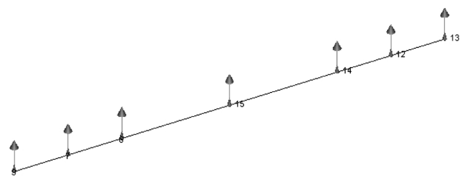

Fig. 18. Measuring points placement on the wooden deal beam – model OMA

The goal of the research (the dynamic analysis was part of a wider research) was to establish if Balzer Hall, the room above the Aula Leopoldina, may be used as a banquet hall. Such a use of the room may be a source of significant dynamic load. The dynamic measurements were performed on one chosen plain girder and the plank immediately below it (Fig. 16).

3.2.2. Modal analysis (OMA)

Using the FDD method (Fig. 19), two eigenfrequencies were identified: 9,7 and 25,8 Hz; using the EFDD, two eigenfrequencies were also identified: 10,14 and 25,68 Hz. The first two eigenforms are shown in (Fig. 20 and Fig. 21) respectively. In the algorithm, it is further possible to identify modal damping, which has been estimated to be, respectively, 14,3% and 1,6%.The relatively large damping for the first mode may be caused by the fact that the movements of separate deals differ in amplitude – their relative movement is therefore big and dry friction appears between them. An imperfection of the anti-symmetry of the second form (which ought to be anti-symmetric) indicates a certain lack of symmetry in the support of the ceiling.

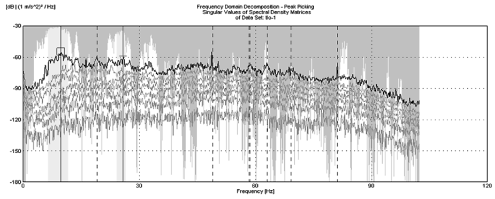

Fig. 19. Singular Values of Spectral Density Matrices

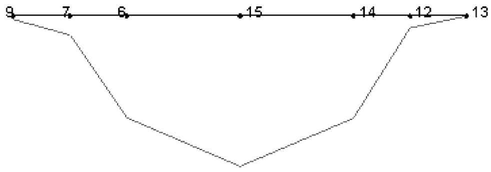

Fig. 20. 1st Eigenshape 9,7 Hz

It was found that the vibration from the "new" ceiling supported on steel plate girders are transferred to the deal beams in their neighbourhood, but are practically not transferred to the deals that are further away. This gives rise to differences in the vibration of the separate deals, which in turn may cause their relative movement. This may result in the appearance of cracks in the plaster ceiling of the Aula Leopoldina, running in a direction parallel to the deal beams. It was therefore recommended that the structure of the floor of the Balzer Hall be separated from the structure of the Aula Leopoldina ceiling.

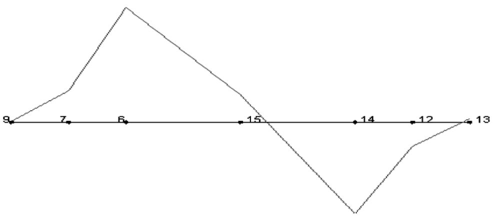

Fig. 21. 2nd Eigenshape 25,8 Hz

3.3. Example 3 – hydropower plant

This example does not deal with a historical building. However, the technology used in this case, which makes it possible to evaluate the crack state of the structure (the depth of the cracks and the possible movement of structural parts separated by cracks), may also be used in historical buildings.

3.3.1. Description of the object and the study

The structure is one of the elements of water accumulation of the water step on the river Wisła. It is made up of three sections with dilatations between them. The structure was built in the 1960s according to a Soviet design. The turbines are also Soviet. Each section is comprised of two hydro complexes. The mass of the generator and turbine with its full equipment is about 883 tons. The turbine shaft is about 1 m in diameter. The underground part of the hydrocomplex consists of the following characteristic parts: inlet spiral, suction pipe and galleries connecting all the hydrocomplexes. These elements are situated below water level and should be waterproof. Unfortunately, the current state of the structure is characterized by the presence of a number of cracks in the concrete of each hydro complex and water leaks through.

3.3.1. Modal analysis

The most important part of the study dealt with suction pipes, where circumference cracks appeared. These cracks could not be diagnosed using any other non-destructive methods due to the presence of water in the pipes. An analysis of eigenforms was proposed as a method of establishing whether the cracks run throughout the thickness of the concrete and whether they work dynamically (the least thickness of the concrete is 2,5 m). (The term „dynamic work of the cracks" describes the possibility of slight relative movement of structure fragments along or across the cracks, as well as the possibility of rotation around each other of the structure fragments separated by the cracks). Discontinuities (breaks and/or jumps) in the eigenform graph, appearing in points on both sides of the crack immediately next to it, indicate that the crack runs throughout the thickness of the concrete and may indicate that the crack works dynamically. In (Fig. 22), the placement of the sensors in the suction pipe Hz6 is shown. Most of the measurements were performed with the use of the seismic accelerometers DeltaTron 8340.

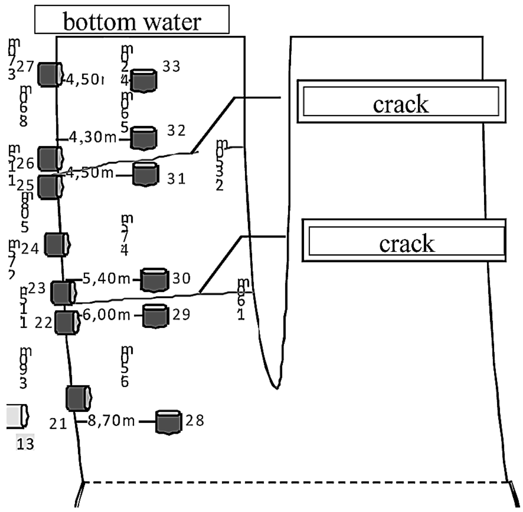

Fig. 22. Measuring points in the suction pipe Hz6 of the hydropower plant

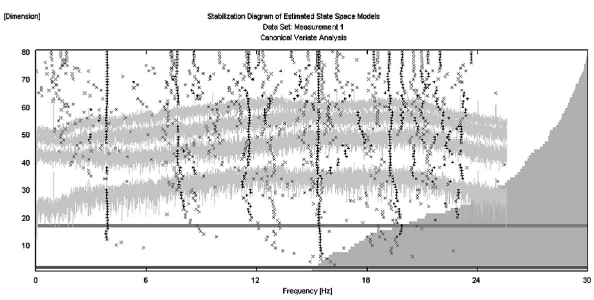

Fig. 23. Stabilization Diagram of SSI (Stochastic Subspace Identification) OMA analysis

Using SSI techniques, the stabilizing diagram for which is shown in (Fig. 23), eigenfrequencies and eigenforms were obtained, shown in (Fig. 24) in green.

In the studied suction pipe Hz6, circumference cracks were found and diagnosed to be running throughout the thickness of the concrete wall of the suction pipe, and to be "working dynamically". These findings were later verified by other, destructive analysis methods.

This example displays the possibility of using OMA techniques to establish the type of damage to a structure. This is a fully non-destructive method that can be used in cases of highly non-homogenous, brittle materials (masonry, stone walls).

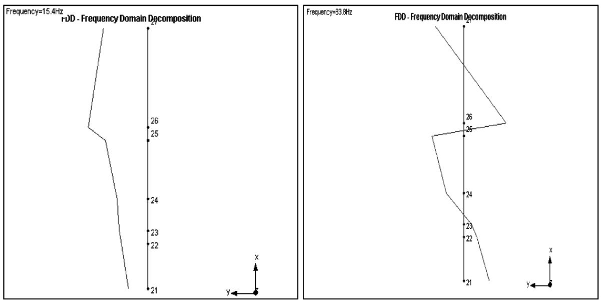

Fig. 4. Eigenforms: first, f1 = 15,4 Hz, and second, f2 = 83,8 Hz

4. CONCLUSIONS

The paper presents contemporary methods and techniques of non-destructive dynamic analyses, which have been chosen with a view to use in historical constructions. The innovation in this approach is the use of two methods of analysis, i.e. theoretical and experimental analyses, to validate and update each other. Emphasis is placed on such analysis methods that ensure a simplified, though not over-simplified, way to achieve a reliable evaluation of the influence of dynamic effects on the behaviour, safety and durability of historical structures and monuments. Simplified methods introduced in Polish and German national standards are described and illustrated with examples of the studied historical structures. The advantages and disadvantages of both approaches are analysed, and conclusions drawn from experience of studies performed according to each are presented.

Alongside the widely used, standard dynamic methods, another approach is also presented: Operational Modal Analysis. OMA is currently the most sophisticated tool for this kind of analysis. This method is used to experimentally study the modal characteristics of a structure during its normal exploitation, without exciting vibration with the use of dedicated devices such as modal hammers or vibration exciters. This is especially advantageous as such "artificial" excitation is difficult to apply to historical structures and may be the source of undue damage to them.

A non-standard, creative method of analyzing the "dynamic work of cracks", developed by the authors with the use of the OMA system, is also presented. Although this approach is illustrated by a study of a non-historical structure (a hydropower plant), it may be used in any structures composed of brittle materials, such as bricks or stonework. It was specifically found that the method is especially useful in establishing whether a crack runs throughout the thickness of the material and whether there is the possibility of relative movement of structure fragments separated by the cracks.

Simple analyses make it possible to observe the studied phenomena quantitatively, without the opportunity to fully evaluate the harmful impact of the vibration on the building.

Simplified analysis procedures introduced by the national standards allow qualitative evaluation, as vibration of a given magnitude may be classified as harmful for the structure. However, these procedures are prone to misunderstandings, as differences in the interpretation of normative expressions occur even among professionals. The use of the non-destructive methods of OMA creates new possibilities of diagnosing damage due to vibration, as it allows a full qualitative evaluation of the analysed structure. In OMA analysis of the structure's vibration, the dynamic influences may be determined on the basis of real dynamic traits of the structure and real forms of vibration. Therefore, the results of the analysis (especially after validation and updating) are fully reliable and do not leave room for differences of interpretation of results. It is for these reasons that OMA provides a wide scope for further development of the diagnostic methods applied to historical constructions.

ACKNOWLEDGEMENTS

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Abstract

The research team from the Structural Dynamics Division of Civil Engineering Institute of Wrocław University of Technology performed experimental dynamic studies of a number of historical constructions. The paper presents the most important aspects of contemporary dynamic analyses of historical structures and is a result of the team's decade-long experience with such analyses carried out with the use of the most modern instruments and technologies. The basis of modern dynamic analysis is the use of three elements: *in situ* dynamic measurements, analysis of the obtained data, and calculation on a FEM model. The interrelations among the three are used to validate and update the FEM model. The methods used in dynamic analyses are very complex and difficult. Aside from them, Polish and German national standards offer simplified methods of evaluating the harmfulness of building vibrations due to ground motions. Three types of analysis methods are presented here: simple dynamic analysis, simplified approaches based on national standards and advanced techniques such as Operational Modal Analysis (OMA). The presented studies and measurements are classified as non-destructive, and are therefore best suited to the analysis of historical structures and monuments. OMA merits special attention, as it yields information on modal frequencies, forms and damping. In comparison to Experimental Modal Analysis (EMA), the greatest advantage of the OMA is that no special excitation of structure vibration is required. In large structures, such excitation is difficult to realize and may lead to local damage, which is especially harmful in historical objects. However, OMA requires more exact and much longer measurements than Experimental Modal Analysis (EMA).

Streszczenie

W pracy opisano najbardziej istotne aspekty współczesnych analiz i badań dynamicznych dotyczących specyficznych konstrukcji jakimi są budowle historyczne. Bazowano na doświadczeniach zebranych w tym zakresie przez pracowników Zakładu Dynamiki Budowli Instytutu Inżynierii Lądowej Politechniki Wrocławskiej. Zaprezentowano nowoczesne metody eksperymentalno-teoretycznych badań dynamicznych tego typu obiektów. W pracy przedstawiono proste analizy zarejestrowanych drgań, podejście normowe (na bazie polskich i niemieckich norm) dotyczące oceny drgań przenoszonych na budynki zabytkowe przez grunt, a także zaawansowaną technikę jaką jest Operacyjna Analiza Modalna (OMA). Stosując OMA można określić parametry modalne badanej konstrukcji i na tej podstawie wnioskować o innych jej własnościach i cechach. Główną zaletą OMA jest możliwość wykonania analizy dynamicznej konstrukcji bez specjalnego wymuszania drgań oraz pomiaru tego wymuszenia. W przypadku konstrukcji budowlanych wymuszanie drgań i pomiary sił wzbudzających są na ogół bardzo kłopotliwe w realizacji ze względu na rozmiary i masę budowli. Dodatkowo, stosowanie młotków modalnych bądź wzbudników drgań jest niewskazane w przypadku konstrukcji zabytkowych, ze względu na możliwość uszkodzenia np. cennych tynków lub malowideł ściennych. Ograniczeniem OMA jest konieczność stosowania precyzyjnych i długich pomiarów, a co za tym idzie, konieczność dysponowania odpowiednim sprzętem pomiarowym i oprogramowaniem. Zdaniem autorów, najwłaściwszym sposobem analizy dynamicznej jest harmonijne połączenie trzech elementów: po pierwsze – pomiarów dynamicznych in situ, po drugie – analiz cech dynamicznych modelu eksperymentalnego konstrukcji z wykorzystaniem danych pomiarowych oraz po trzecie – wykonanie niezależnych obliczeń konstrukcji z zastosowaniem jej modelu MES. Wzajemne walidowanie wyników analiz doświadczalnych i teoretycznych oraz otrzymanie jak największej ich zgodności prowadzi ostatecznie do otrzymania wiarygodnych i praktycznie przydatnych wniosków i zaleceń wykonawczych.

Maria Cristina Forlani[1], Donatella Radogna[2], Alberto Viskovic[3]

Wooden framed structures for masonry buildings retrofitting. A pilot project in Caporciano

Drewniane konstrukcje ramowe w naprawach budynków murowanych. Pilotażowy projekt w Caporciano

Keywords: Masonry retrofitting, Reuse, Mix Structural System, Wooden structures

Słowa kluczowe: uzupełnienie konstrukcji murowanych, ponowne użycie, mieszany system konstrukcyjny, konstrukcje drewniane

1. INTRODUCTION

The study refers to the masonry buildings ruins rehabilitation in the post earthquake of Caporciano (L'Aquila, Italy). The philosophy at the bottom of the reconstruction plan wants to rehabilitate the buildings, in case their integrity is acceptable.

In case the existing buildings are ruins with an evident evocative character, we think it is desirable to strengthen them by a partial transformation – with contemporary technologies – with the purpose of re-establishing their use.

Fig. 1. The state of conservation of the existing masonry buildings

For the existing building rehabilitation we propose wooden systems. The proposal comes from the aim of keeping and conserving the heritage, but without making historic fakes reconstructions. This is a way for integrating the "memory" with the new buildings in a harmonic relation and testifying the necessary evolution for satisfying the current life an environment needs.

For bestowing new use possibilities on ruins, we do not propose an existing buildings reconstruction but a construction in the existing buildings.

This construction assumes also the role of defence against new seismic stresses.

The plan proposes a construction on the existing building. The intervention choices admit the opportunity of "repairing the damages", "leaving things almost unchanged", that is to say using sustainable constructive materials and systems, that are compatible with the existing buildings characteristics even if different and "transfer".

2. THE RUINS REHABILITATION THROUGH WOODEN SYSTEM INTEGRATION

The building systems rehabilitation, we designed, foresees the integration of light and removable systems in the conservation of the masonry systems "ruins". The wooden systems have the double role of making the buildings re-usable as well as to lend them security.

We propose the "insertion" of self-bearing volumes in the existing masonry walls, that are able to consolidate and stabilizing the existing structures (through horizontal wooden and iron system). The new volumes have wooden and iron bearing framework and wooden and glass closures.

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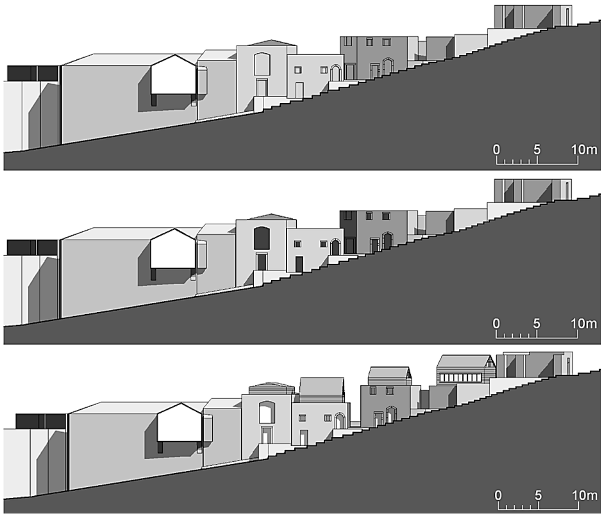

Fig. 2. Front view of the existing buildings, the removed parts (in red) and the insertion of the new wooden buildings

These new volumes can exploit as much as possible the climatic factors (wind and sun) for internal spaces natural ventilation, illumination and acclimatization.

So the new volumes can supply the lost performance levels, above all for what concerns the safety, well-being, usability, management and aspect. In this direction, we reassert the importance of satisfying the intervention reversibility needs (together with those of sustainability and safeguard). The reversibility was recognised as a fundamental character of the human activities (such as to make architecture) of our age.

Planning and building according to the reversibility paradigm means to admit the inversion possibilities of the constructive process: from the realization to the "zero remains" disposal.

The reversibility of the constructive sequences in the building processes, indeed, means above all the possibility of recovering constructive materials and elements as well as of repairing almost totally the intervention ex-ante conditions. So reversibility means not transforming in an irreparable way the existing masonry systems.

The considered settlement of the intervention lies in Caporciano, in a midway position between the old town hill and the plain expansion area. The settlement consists in some masonry walls part of an original multi-storey (2 or 3) system; the current condition of this system, now show ruined attics and roofs as well as missing casings.

We chose to strengthen the existing walls and to settle (inside the perimeter given by the old walls) a new wooden framed and wood fibre insulated building. The roofs, conveniently oriented, will be able to produce renewable energies (thermal solar and photovoltaic systems) and the plants will lean to minimize the water consumption, recovering rainwater and recycling grey waters, as well as to hold back the other consumption.

In synthesis the proposal comes from the following objectives:

- the answer to the function specific needs linked to new and traditional family units,
- the environmental sustainability general needs,
- the choice (fundamental for the sustainability objectives) of rehabilitating the existing buildings.

3. THE ENVIRONMENTAL SUSTAINABILITY NEEDS AND THE WORKS REQUIREMENTS AND PERFORMANCE

We faced the emissions, the waste production and the resources consumption curb, considering the building materials, the water and the energy.

The choice of working on the existing buildings with a rehabilitation and renovations project guarantees the soil consumption curb. So the existing buildings rehabilitation and strengthen costs are balanced out by the saving due to the avoided ruins demolition and debris storage works (that involve also environmental impact and transport to dump costs).

The polluting emission curb is given by the reduction of the CO_2 production. The proposed constructions are Class A certified thanks to the use of more efficient plants, against the traditional ones and the use of renewable energies and certified materials.

For the reduction of waste production we considered principally the building and demolition waste which are one of the most polluting source. SO we decided to rehabilitate the ruins that otherwise would have become, if demolished, a big quantity of debris causing a big environmental impact and dump problems.

For what concerns the building materials, we considered natural and renewable resources with low energy worked in. According to the zero Kilometres philosophy, for the vertical and horizontal closures, we chosen technological systems made by industries situated as closer as possible against the yard.

For what concerns the drinkable water saving we foresee rainwater collecting plant and a grey water management system. The rainwater recovery and the grey water management allow to minimize the drinkable water consumption and to have a quantity of water, which can be used outdoor (as for irrigation).

Fig. 3. The insertion of the wooden volumes in the existing ruins

For what concerns the energy saving we used, as renewable energetic sources, solar panels (26 square metres of vacuum packed panels) for the sanitary warm water and for the heating, and photovoltaic panels (50 square metres), for the electricity production.

The thermal solar plant concurs for the 70% to satisfy the sanitary warm water and the heating need. The remaining 30% energy need is satisfied by pellets (in the future supposition of a local production from the woody and farm system management) or methane gas hot-water heating. The photovoltaic plant satisfies over the 50% domestic electric energy need.

For satisfying the usability needs, the existing building dimensions allows to organize six accommodations for twelve persons (with the annexed services), suitable for guaranteeing the spaces comfort and quality. The access to the area and the ways are adequate to be covered by baby carriage and disabled wheelchair.

For satisfying the well-being needs, thermal comfort and the natural illumination are the most considered elements in the project. We considered the climatic resources for complementing the heat intake, favouring the ventilation and implementing the natural illumination thanks to bio-climatic device.

Fig. 4 The masonry buildings before and after the rehabilitation works

The plants were conceived to define a system integrating renewable energetic and traditional heating plants. For better optimizing the heating production and management, we propose the installation of a centralized plant using a floor radiant system so to improve considerably the energetic efficiency of the building-plant system.

For satisfying the management needs, the chosen dry-stone systems facilitate ordinary maintenance works and, in particular, allow configuring sustainable life end strategies (reuse, rehabilitation and components recycling).

Other important requirements defined with the design are the spaces adaptability and flexibility. The facility of displacement of the partition walls allows obtaining different accommodation typologies.

Also the plants are flexible that is to say that the possible elimination or different positioning of some toilet facility do not requires destructive works but just dismantling works.

For what concerns the satisfaction of the integration needs, the choice of rehabilitating the ruins comes from the will of conserving the patrimony without making reconstruction of historical fakes.

Therefore the project consists in the integration of the “historical memory” with the new building in a harmonic relation and showing the necessary evolution for satisfying the current life and environmental needs.

For what concerns the satisfaction of the environmental safeguard needs, we selected the building materials searching the as smaller as possible impact.

Anyway the objective is to implement the local conditions proposing the use of local products and the technological systems simplification above all in the use of materials with supply short weaving factory.

For what concerns the satisfaction of the aspect needs, after the works the ruins acquire a complete image.

We suggest realizing systems able to restore the morphologic and dimensional integrity of the existing buildings. So we transform the collapse perception reality in memory.

The distinction between the ruins and the new constructions is immediately thanks to the strong difference between the original and the new materials and constructive systems. On the other hand the overall image results uniform because we do not alter the local buildings volumes traditional shapes and dimensions.

4. THE MASONRY RUINS STRUCTURAL RETROFITTING

4.1. Introduction

As regards the safety demand, the structural typology chosen for the residential buildings and the social service buildings is a mixed system: the old survived masonry ruins (opportunely refurbished and consolidated) connected and collaborating with new wooden 3D box-framed structures.

The old and damaged masonry structures are consolidated through local reparations, local little demolitions and reconstructions, mortar injections and a layer (40-50 cm of vertical thickness) of reinforced new masonry (reinforced by means of steel or PVC nets placed inside the horizontal mortar joints) on the free top of the masonry walls. The complementary wooden structure, connected to the masonry, guarantees structural box behaviour in case of seismic actions. As it is well known, for the seismic behaviour of masonry buildings, it is necessary to guarantee that the structure works as a closed box, in such e way to redistribute the horizontal loads among all the vertical masonry panels, without to leave some of these panels working alone and overloaded along its own weaker principal axis.

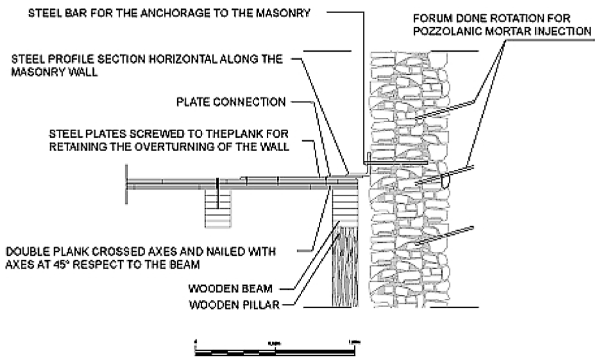

Fig. 5. The link between a masonry wall and the wooden frame at a floor level

What is more, to develop a box behaviour means that the floors stiffening, in their horizontal own plane, avoiding relative deformations of the horizontal plans at the upper levels, improves the structural collaboration among the walls in such a way to improve the global torsion resistance; this structural need arise as torsion effects are always present (especially in existing buildings non specifically designed taking into account the seismic actions) because of the not regular masses distribution in relation to the global horizontal shear stiffness centre.

4.2. The pilot project

In this pilot project the closed box behaviour is obtained through trussed elements in all the horizontal panels of each level, in the roof level, inside the vertical wooden frames and by the connection among those horizontal and vertical wooden panels and frames.

The trussed elements are made by steel ties or by wood; in the case of the wooden floors the horizontal shear stiffness

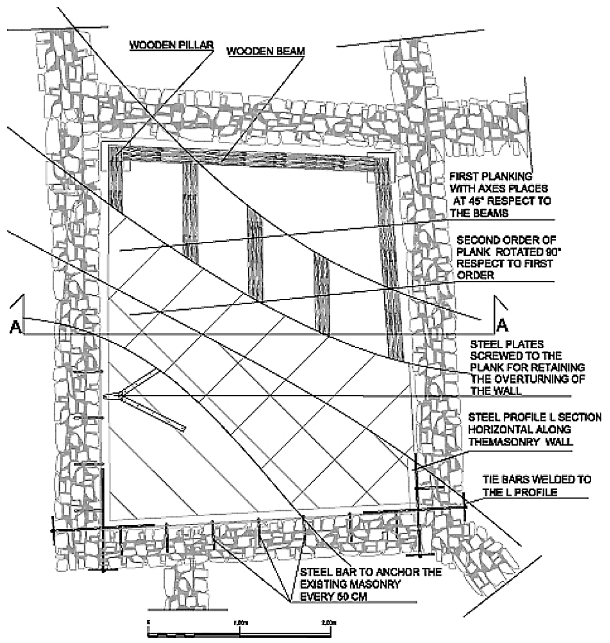

Fig. 6 .A typical floor plan view with the double layer of crossed planks and the connections between frames and masonry

is provided by the over position of two or three horizontal layers of planking placed in cross directions, fastened each to the other and connected to the vertical frames as reported in Figures 5 and 6. The wooden 3D trussed frame is connected to the masonry walls by means of steel plates (or bars) fixed to the masonry using steel stirrups.

A similar connection to the masonry is provided at the roof level also, in those cases where the survived walls are standing up with their total original height as reported in Figure 7. At the roof level it is the roof itself that provides the horizontal shear stiffness, thanks to a double layer of crossed wooden planking.

In addition to the retrofitting for the bending actions orthogonal to their own main plane, the masonry walls are reinforced also for shear actions parallel to their own plane, using trussed vertical wooden frames placed parallel and near to each masonry wall.

These vertical shear resistant elements are built by wooden frames trussed by diagonal steel bars.

The vertical trusses collaborate with the masonry walls up to the top of the ruins and work alone in the upper part of the new wooden structure, at the levels where the masonry is no more present.

As regard the shear actions parallel to the principal plan of each wall, the collaboration between the new wooden structure and the masonry walls is mainly significant during the non linear behaviour before the ultimate load of collapse: this because of the high shear stiffness of the not damaged masonry walls limits the new structure work, in case of not very strong horizontal actions.

On the contrary, as regard the bending and overturning actions orthogonal to each masonry wall main plane, the collaboration between the two structural systems arise earlier, with lower values of horizontal actions, because of the lower ultimate load of a masonry wall subjected to transversal overturning actions, respect to the case of shear actions parallel to the wall itself.

Thus masonry walls are maintained in their vertical positions by the trussed wooden frames and the links between these two structures are placed at each horizontal level.

Each horizontal shear stiffened floor is completed by means of steel "L" or "T" profiles fixed (screwed) to the double planking, all along the perimeter of each floor (Figures 6 and 7), like horizontal frames. These steel profiles are then also connected to the masonry every 50 cm all along their length.

These steel frames works also as horizontal chains, connecting the masonry walls along the principal directions of each building.

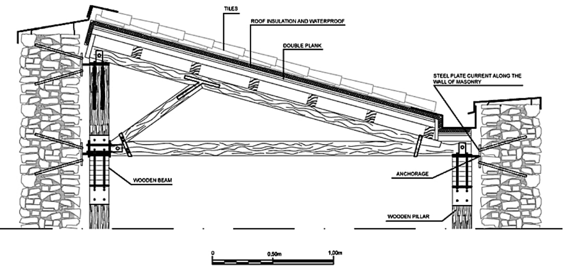

Fig. 7. An example of roof structure with the connection to the masonry, in a case where the masonry walls have their original elevation

For this purpose and also for the purpose to obtain a strong connection of the floors with the masonry corners, the steel profiles are welded to steel tie bars passing through the masonry at each floor corner, as reported in Figure 6.

The new wooden frames have a foundational structure which works also as an improvement and an enlargement of the original masonry walls foundation.

In Figure 8a it is reported the first phase of the building up of the new structure, with the consolidation of the existing foundational structure and the digging of the space for the new foundation. In Figure 8b it is reported the new structure with the foundation linked and integrated with the original one.

As regard the original masonry structures, the parts not very damaged are consolidated with pozzolanic mortar injections, while the more damaged are repaired with partial dismantling and rebuilding with the technique of the "reinforced" masonry.

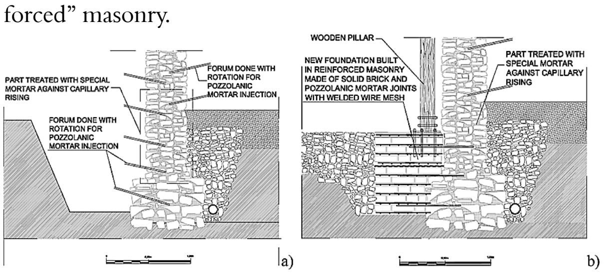

Fig. 8. The restoration of the masonry foundation (a) and the new wooden frame foundation (b)

That technique, in the case of restoration of existing masonry structures, consists in the use of the original stone blocks (or blocks recovered from the collapsed portions of the structure), in the use of pozzolanic fibre-reinforced mortar and in the placing a thin steel net (made by wires with no more than 2 mm of diameter) or a PVC net (fibreglass reinforced) inside some of the horizontal mortar joints. Generally the reinforcing net has to be placed every two or three horizontal joints along the vertical direction.

This is the same technique used for the reinforcement of the top of the masonry structure partially collapsed, with the aim to structurally "close" (and to protect to the weathering) the free top boundary of the wall. However the same technique is used also on the top of those walls still standing with their original height, in such a way to reinforce the link with the roof structure.

The new wooden structures, together with all their steel elements and details, are designed and verified in such a way to resist without serious damages to the design seismic action provided by the Italian Code for the pilot project site, which is inside the Caporciano Municipality area, where they have to be considered horizontal seismic ground accelerations in the order of $a_g = 0, 265$ g.

5. CONCLUSIONS

The proposed projects wish to suggest an interventions realisation aiming to guarantee a local sustainable development also through the choice of appropriate building materials and systems.

For this reason, we want to ward off either abandonment phenomena or existing buildings inaccurate transformation works, considering the contemporaneous life styles interests. That is to say, we define the possibility of revitalising the considered place, settling new activities linked to ancient traditions, safeguarding the environment identifying characters. So we want to avoid the risk of transforming the buildings (the ruins) in the "themselves museum". Our proposal represents an application example of how the technology can put into communication ancient and contemporaneous elements (above all for what concerns building techniques and materials and functions).

The developed work tries to interpret and make concrete the "new conservation science" principles in the satisfaction of the current sustainability needs, so that the safeguard and transformation interests find common objectives.

The strengthening of the ruins is integrated in the realization of new wooden systems then the masonry traditional architecture uses the technological innovation of the wooden system for being used again. So we fade the limits outlines between the existing building and the new building design.

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Abstract

The paper deals with the use of wooden framed structures for the retrofitting and rebuilding of partially destroyed masonry buildings. When the damages are not so large to require a complete demolition but they are also not so localized to permit simple reparations, there is always the problem: how much to rebuild without to obtain an historic fake?

The philosophy at the base of the Rehabilitation Plan for Caporciano (a municipality in the L'Aquila earthquake area) follows two different ways: a) the demolition, when the ruins envelop is less than the 30% of the original building volume and/or the masonry is too much damaged to be consolidated; b) the consolidation of the masonry and their stabilization through the collaboration with new wooden structures built inside the free spaces leaved by the collapsed roofs and floors.

The choice to recover, in such a way, the survived existing structures, descend from the aim to conserve the historical heritage without rebuilding historic fake but integrating the "memory" with the "new" in an harmonic collaboration, testimonial of the necessary technological evolution for the life and environment contemporary needs satisfaction.

The project deals with a new structure inside the ruined walls with two goals: the consolidation and the stabilization of the survived walls and the building of new and efficient wooden volumes, with wooden fibres thermal insulation, inside the perimeters of the masonry walls.

Streszczenie

Artykuł omawia wykorzystanie drewnianych konstrukcji ramowych w naprawach i odbudowie częściowo zniszczonych budynków murowanych. Jeśli uszkodzenia nie są aż tak poważne, aby wymagały całkowitej rozbiórki obiektu, ale też nie tak nieistotne by wystarczyły zwykłe naprawy, zawsze powstaje problem: ile należy odbudować by uniknąć historycznych zafałszowań?

Filozofia leżąca u podstaw Planu Odnowienia Caporciano (miasta w rejonie L'Aquila nawiedzonym przez trzęsienie ziemi) pozwala na dwa różne rozwiązania: a) wyburzenie, gdy zniszczone jest mniej niż 30% oryginalnej substancji i/lub mury są zbyt zniszczone b) konsolidację elementów murowanych i ich stabilizację poprzez wprowadzenie nowych elementów konstrukcyjnych z drewna, wzniesionych wewnątrz wolnych przestrzeni pozostawionych przez zniszczone sklepienia i podłogi.

Decyzja by w ten sposób odrestaurować to, co przetrwało, ma na celu zachowanie historycznego dziedzictwa bez fałszowania zabytku, ale dzięki harmonijnemu zintegrowaniu "starego" z "nowym", co jest świadectwem nieuniknionej ewolucji technologicznej koniecznej by zaspokoić współczesne potrzeby życia i środowiska.

Projekt wprowadza nową konstrukcję do wnętrza zniszczonego budynku z dwóch powodów: w celu skonsolidowania i ustabilizowania ocalałych ścian, oraz stworzenia nowej, skutecznej przegrody drewnianej wypełnionej izolacją termiczną z wełny drzewnej, wewnątrz oryginalnych kamiennych murów.

Maciej Nocoń*

Renowacja murów ceglanych oraz kamiennych przy użyciu materiałów quick-mix oraz Tubag

Renovation of brick and stone walls using quick-mix and Tubag materials

Renowacja ceglanych murów twierdzy w Srebrnej Górze

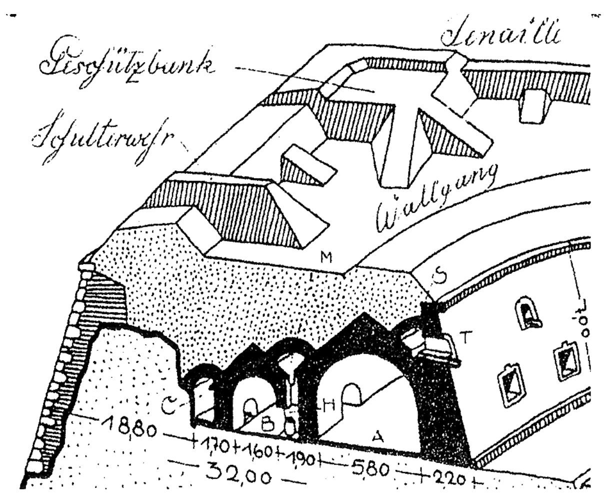
Większość turystów odwiedzających twierdzę w Srebrnej Górze zastanawia się, dlaczego tak olbrzymia warownia powstała właśnie w tym miejscu. Aby odpowiedzieć na to pytanie należy cofnąć się o 250 lat. Śląsk był wówczas areną krwawych zmagań pomiędzy Prusami i Austrią. W efekcie trzeciej wojny śląskiej (1756-1763) zwanej wojną siedmioletnią, wojskom pruskim udało się odzyskać utracone na rzecz Austrii ziemie. Stałe zagrożenie i bolesne doświadczenia wojenne skłoniły króla pruskiego Fryderyka II Wielkiego do ufortyfikowania przejścia górskiego pomiędzy Śląskiem a hrabstwem kłodzkim. W 1763 r. pruski inżynier wojskowy pułkownik Ludwig Wilhelm Regler wykonał pierwsze prace pomiarowe w miejscu przeznaczonym na budowę twierdzy. Rok później powstał projekt zakładający budowę rozległego zespołu fortecznego, którego centralnym miejscem miał być donżon – wieloboczna reduta otoczona czterema niższymi wieżami cylindrycznymi. W 1765 r. rozpoczęto realizację tego ambitnego przedsięwzięcia. Budowę twierdzy wraz z przyległymi bastionami, fortami, fosami, redutami, wysokimi na trzydzieści metrów murami ukończono po 20 latach w 1785 r. W samym donżonie znajduje się około stu pięćdziesięciu kazamt. Pełniły one funkcje lazaretu, pomieszczeń komendantury, browaru, piekarni, oraz różnego rodzaju magazynów. Przechowywano w nich między innymi 280 ton prochu strzelniczego. Twierdza w założeniu budowniczych miała być samowystarczalna, mogła przetrwać oblężenie trwające nawet do pięciu miesięcy.

Twierdza srebrnogórska była jednym z najważniejszych elementów obrony państwa pruskiego przed głównym wrogiem jakim była Austria rządzona przez Habsburgów. Doświadczenia późniejszych wojen napoleońskich zmieniły jednak doktryny wojenne oraz układ sił. Znaczenie obronne twierdzy stopniowo malało. Twierdza pełniła swą militarną rolę przez ok. 100 lat. Po demilitaryzacji donżon stał się jedną z większych atrakcji turystycznych Srebrnej Góry. W roku 1885 wewnątrz twierdzy powstała pierwsza restauracja dla obsługi ruchu turystycznego.

W roku 2003 powołano do działania Forteczny Park Kulturowy, którego zadaniem jest renowacja twierdzy oraz ochrona jej warownego krajobrazu kulturowego. W efekcie tych działań w roku 2006 rozpoczęto pierwsze prace konserwatorskie, które trwają do chwili obecnej.

Od momentu wybudowania załoga twierdzy uskarżała się na chłód i wilgoć panujące w kazamatach. Niezdrowy mikroklimat spowodowany był oczywiście przez wodę przesączającą się przez sklepienia. Na rycinie 1 pokazano przekrój przez koronę donżonu. Widoczne są trzy rzędy sklepionych kazamat przysypanych wielometrową warstwą ziemi. W nasypie ziemnym budowni-

Fot. 1. Donżon twierdzy w Srebrnej Górze
Photo 1. Donjon of the fortress in Srebrna Góra



Ryc. 1. Przekrój przez koronę donżonu, widoczna sieć kanałów drenażowych nasypu ziemnego nad kazamatami
Fig. 1. Cross-section of the Donjon top, visible network of drainage ditches in the embankment above the dungeons

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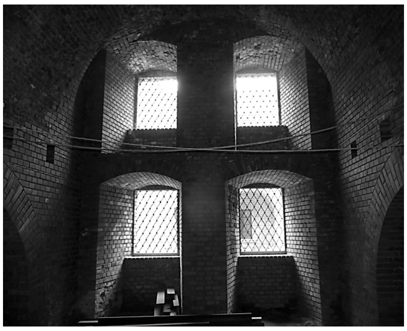
Fot.2. Wnętrze kanału drenażowego odwadniającego nasyp ziemny
Photo 2. Interior of the ditch draining the embankment

Fot. 3. Elewacja donżonu, widoczny wylot kanału drenażowego wraz z charakterystycznym zawilgoceniem muru
Photo 3. Elevation of the Donjon, visible mouth of the drainage ditch with characteristic damp patches on walls

Fot. 4. Elewacja donżonu przed renowacją
Photo 4. Elevation of the Donjon before renovation

Fot. 5. Elewacja donżonu po renowacji, zastosowano zaprawę murarską TZM-s
Photo 5. Elevation of the Donjon after renovation, TZM-s mortar was applied

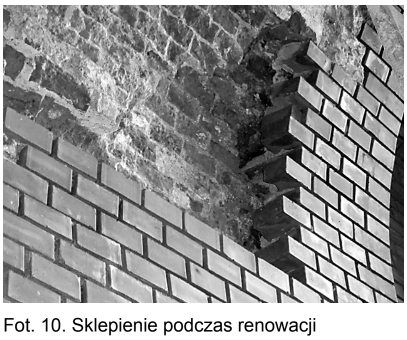
Fot. 6. Ściana przed renowacją, widok od wewnątrz
Photo 6. Wall before renovation, view from inside



Fot. 7. Ściana po renowacji, widok od wewnątrz. Zastosowano zaprawę murarska TZM-s oraz fugę TKF
Photo 7. Wall after renovation, view from inside. TZM-s mortar and TKF plastering grout were applied

Fot. 8. Ściana podczas renowacji
Photo 8. Wall during renovation

Fot. 9. Fragment ściany po renowacji, widoczne połączenie nowego muru ze starym
Photo 9. Wall fragment after renovation, visible borderline where the new wall joined the old



Fot. 10. Sklepienie podczas renowacji
Photo 10. Vault during renovation

Fot. 11. Sklepienie podczas renowacji
Photo 11. Vault during renovation

Fot. 12. Łuk ceglany przed renowacją
Photo 12. Brick arch before renovation

Fot.13. Łuk ceglany podczas renowacji
Photo 13. Brick arch during renovation

Fot.14. Łuk po renowacji
Photo 14. Brick arch after renovation

Fot. 15. Sklepienie po iniekcji cementowo-trasową zaprawą iniekcyjną TZV-p, widoczna rysa zasklepiona płynną zaprawą
Photo 15. Vault after being injected with the cement-trass mortar TZV-p, visible crack sealed with the liquid mortar

czowie wykonali sieć kanałów drenażowych (fot. 1), których zadaniem było odprowadzenie wód opadowych z nasypu. Wyloty kanałów można dostrzec na elewacji w postaci małych okienek obramowanych elementami z piaskowca (fot. 2). Oczywiście taki sposób odwodnienia nasypu ziemnego był niedoskonały – sklepienia oraz ściany były stale zawilgocone. Woda przesączająca się przez sklepienia wypłukiwała zaprawę, natomiast zawarte w niej rozpuszczalne sole powodowały powolną ale systematyczną destrukcję ceglanych ścian oraz sklepień. Stopień zniszczenia murów w momencie rozpoczęcia prac renowacyjnych widać na fot. 3, 5, 7. Natomiast na fot. 4, 6, 8 pokazano te same elementy po zakończeniu renowacji.

Od początku prac renowacyjnych na placu budowy stosowane były specjalistyczne materiały produkowane przez firmę quick-mix oraz Tubag. Prace remontowe rozpoczęto od odtwarzania i wzmacniania poważnie osłabionych murów. Do ich odbudowy wykonawcy stosowali między innymi zaprawę trasowo-cementową TZM-s. Zaprawa ta jest przeznaczona do wznoszenia historycznych murów ceglanych oraz kamiennych narażonych na stałe oddziaływanie wody. W wielu miejscach grubość odtwarzanych murów wynosiła kilkadziesiąt centymetrów, powodowało to znaczne zużycie zaprawy, a co za tym idzie, duże koszty. Firma quick-mix opracowała dla potrzeb tego obiektu receptury zapraw murarskich przygotowywanych na palcu budowy. Zaprawy te wytwarzano z lokalnego kruszywa przy użyciu cementu trasowego Tubag TZ-o oraz wapna trasowego Tubag TK. Po zakończeniu prac murarskich, nowe mury zostały wyspoinowane trasowo-wapienną zaprawą do spoinowania TKF. Zgodnie z zaleceniami konserwatorskimi zaprawa do spoinowania była pigmentowana na charakterystyczny kolor. Celem tego zabiegu było pokazanie starego i nowego wątku muru ceglanego (fot. 8).

Po odbudowie i wzmocnieniu murów przystąpiono do renowacji ceglanych łuków oraz sklepień. Zakres prac murarskich pokazano na fot. 9, 10, 11, 12, 13. Prace te przeprowadzono używając do murowania trasowo-cementową zaprawę TZM-s, natomiast do fugowania trasowo-wapienną zaprawę do spoinowania TKF. Sklepienia, podobnie jak mury, przez wiele lat narażone były na oddziaływanie przesączającej się wody. W efekcie konstrukcja sklepień została znacznie osłabiona, wypłukaniu uległa zaprawa murarska, pojawiły się liczne spękania. W wielu miejscach szerokość rys dochodziła do 30 mm, groziło to katastrofą budowlaną. Do wzmacniania osłabionych sklepień zastosowano cementowo-trasową zaprawę iniekcyjną TZV-p. Zaprawa TZV-p została wtłoczona w strukturę sklepień poprzez specjalnie nawiercone otwory. Zaprawę tłoczono za pomocą pompy ślimakowej. Płynna, bezskurczowa zaprawa TZV-p wypełniła puste przestrzenie, kawerny oraz rysy znajdujące się wewnątrz sklepień (fot. 14). W sklepienia wtłoczono łącznie ok. 10 ton zaprawy. W efekcie udało się odtworzyć pierwotną nośność sklepień.

Renowacja twierdzy w Srebrnej Górze trwa już od kilku lat, związana jest ze znacznymi wydatkami. Stanowi jednak niewątpliwy magnes przyciągający coraz liczniejsze grupy turystów odwiedzających ten uroczy i tajemniczy zakątek Dolnego Śląska.

Fot. 16. Brama Myśliborska oraz fragment muru obronnego w Trzcińsku--Zdroju
Photo 16. Myśliborska Gate and a fragment of the defensive wall in Trzcińsko-Zdrój

Fot. 17. Mury obronne przed renowacją
Photo 17. Defensive walls before renovation

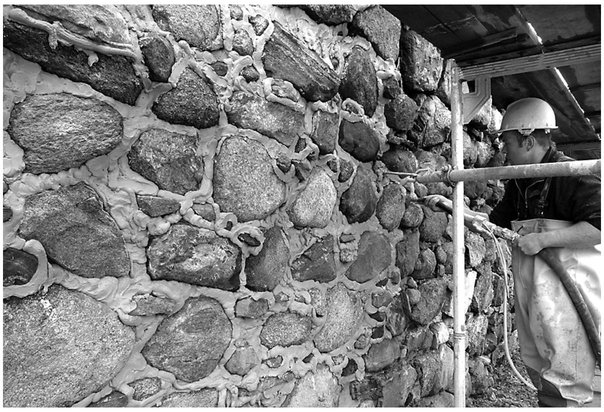
Renowacja kamiennych murów obronnych w Trzcińsku-Zdroju

Podczas wakacji rzesze turystów przemierzają szlaki komunikacyjne wiodące z południa Polski nad Bałtyk. Nowy odcinek drogi ekspresowej z Gorzowa do Szczecina zachęca do szybkiej jazdy. Warto jednak kilkadziesiąt kilometrów za Gorzowem nieco zwolnić, skręcić w kierunku zachodniej granicy i na chwilę odwiedzić Trzcińsko-Zdrój. Jest to niewielka miejscowość leżąca na uboczu głównych szlaków komunikacyjnych. W X wieku była tu osada rolniczo-rybacka. Po opanowaniu tych ziem przez margrabiów brandenburskich osada zostaje przebudowana i otrzymuje obecny układ urbanistyczny. Miasto w swoich burzliwych dziejach znajdowało się w latach 1402-1454 pod panowaniem Zakonu Krzyżackiego, w roku 1433 zostało spalone przez tabory husyckie. Znaczne zniszczenia pozostawiła po sobie wojna trzydziestoletnia, podczas której miasto było pustoszone zarówno przez armię cesarską, jak i szwedzkie wojska Gustawa Adolfa. W końcu XIX wieku odkryto pokłady borowiny, wybudowano zakład leczniczy – miasto stało się Zdrojem.

Trzcińsko-Zdrój jako jedno z niewielu miast w Polsce posiada, zachowane niemal w całości, średniowieczne fortyfikacje obronne. W skład fortyfikacji wchodzą mury miejskie o łącznej długości ponad 1400 m, dwie bramy: Myśliborska oraz Chojeńska, dwie baszty: Prochowa i Bociania, jak również strażnice. W XIX wybito w murach 40 bramek zamykanych furtkami, bramki te miały ułatwić komunikację mieszkańcom miasteczka. Mury obronne zostały wzniesione na początku XIV wieku z granitowych głazów narzutowych oraz kamienia polnego. Mają grubość ok. 1 m, pierwotnie wznosiły się na wysokość od 6 do 9 m. Inwentaryzacje murów wykonane w latach

Fot. 18. Usuwanie pozostałości po mocnych zaprawach cementowych
Photo 18. Removing remains of strong cement mortars

Fot. 19. Mycie muru wodą pod ciśnieniem
Photo 19. Clearing the wall with pressurized water.



Fot. 20. Spoinowanie muru zaprawą TKF, aplikacja mechaniczna
Photo 20. Sealing joints in the wall with TKF mortar, mechanical application

Fot. 21. Po ok. 1-2 godzin można przystąpić do dalszej obróbki zaprawy TKF
Photo 21. After app. 1-2 hours, TKF mortar can be further processed

Fot. 22. Warstwa spadkowa na koronie muru z zaprawy murarskiej TWM
Photo 22. Sloping layer on the wall top from TWM masonry mortar

Fot. 23. Dodatkowe zabezpieczenie korony muru, warstwa cegieł na zaprawie murarskiej TWM
Photo 23. Additional protection for the top of the wall, a layer of bricks on masonry mortar TWM

Fot. 24. Mury po renowacji
Photo 24. Walls after renovation

Fot. 25. Mury po renowacji
Photo 25. Walls after renovation

ubiegłych jednoznacznie wskazywały na ich zły stan techniczny. Remonty przeprowadzone w latach 60. ubiegłego wieku przyniosły murom więcej szkód niż pożytku. Do remontów wykorzystywano bowiem mocne zaprawy cementowe (fot. 17) o niewłaściwych parametrach fizycznych oraz mechanicznych. Zaprawy cementowe mają małą zdolność kapilarnego podciągania wody oraz wysychania. Ich wytrzymałość mechaniczna oraz współczynnik rozszerzalności liniowej są zupełnie inne od oryginalnej, wapiennej zaprawy murarskiej. Dodatkowe zniszczenia spowodowane zostały przez korzenie krzewów. Rośliny znalazły dogodne warunki do rozwoju w szczelinach muru, w których przez lata gromadziły się różnorodne szczątki organiczne.

W roku 2012 przystąpiono do kompleksowego remontu fortyfikacji otaczających miasteczko. W najbardziej zniszczonych i uszkodzonych fragmentach remont polegał na rekonstrukcji murów oraz baszty. Do murowania użyto kamieni polnych układanych na **trasowo-wapiennej zaprawie TWM**. Spoiwem tej zaprawy jest wapno trasowe Tubag. Zaprawa TWM ma wytrzymałość zbliżoną do oryginalnej zaprawy wapiennej, produkowana jest na bazie kruszywa o uziarnieniu od 0 do 4 mm. Zalecana jest szczególne do renowacji zabytkowych murów ceglanych oraz kamiennych.

Po odtworzeniu brakujących elementów fortyfikacji miejskich mury zostały zafugowane **trasowo-wapienną zaprawą do spoinowania TKF**. Spoiwem tej zaprawy jest również wapno trasowe TUBAG. Zaprawa ma wytrzymałość zbliżoną do oryginalnej zaprawy wapiennej, produkowana jest na bazie kruszywa o uziarnieniu od 0 do 1,2 mm. Zaprawa TKF ze względu na niski skurcz, wysoką nasiąkliwość, szybkie wysychanie oraz niski opór dyfuzyjny zalecana jest do spoinowania zabytkowych murów ceglanych oraz kamiennych. Do zaprawy TKF podczas mieszania można dodawać lokalne kruszywa o uziarnieniu od 4 do 8 mm, w efekcie uzyskuje się zaprawę nie tylko wytrzymałościowo, ale i wizualnie zbliżoną do oryginału.

Prace związane z fugowaniem rozpoczęto od usunięcia pozostałości po „remontach" z lat 60., czyli od skucia mocnej zaprawy cementowej. Roboty te wykonywano ręczne jak również mechaniczne (fot. 18). Mury zostały następnie zmyte wodą pod ciśnieniem, podczas mycia mury nawilżono i przygotowano do fugowania. Kolor zaprawy do fugowania dopasowano do koloru oryginalnej zaprawy wapiennej. Uzyskano go metodą prób w wyniku zmieszaniu 25 części wagowych fugi TKF w kolorze białym z 1 częścią wagową fugi TKF w kolorze żółtym. Oba kolory fugi najpierw starannie mieszano ze sobą „na sucho" w mieszarce wolnospadowej. Następnie dodano wodę i jeszcze raz wymieszano do momentu uzyskania jednolitej barwy. Ze względu na duże powierzchnie murów i napięty harmonogram prac remontowych zdecydowano się na fugowanie metodą mechaniczną. Zaprawa do fugowania podawana była wężami za pomocą specjalnie przystosowanej do tego celu pompy. Po ułożeniu zaprawy w fugach i odczekaniu, w zależności od warunków atmosferycznych, od 1 do 2 godzin powierzchnie fug były obrabiane ręcznie w celu nadania im właściwego kształtu oraz faktury (fot. 21). Korony murów zabezpieczono przed wnikaniem wody pochodzącej z opadów atmosferycznych wykonując warstwę spadkową z **trasowo--wapiennej zaprawy TWM** (fot. 22). W wybranych partiach murów na warstwie spadkowej ułożono dodatkową okładzinę ochronną z cegieł (fot. 23) murowanych na **trasowo-wapiennej zaprawie TWM**.

Obecnie prace renowacyjne dobiegają do końca, stare mury nabierają stopniowo dawnego blasku. Jednocześnie tereny przylegające do murów obronnych odzyskują swoją świetność, stają się atrakcyjną przestrzenią zachęcająca do rodzinnych spacerów zarówno dla mieszkańców Trzcińska-Zdroju jak i turystów odwiedzających malownicze miasteczko.

Streszczenie

W artykule opisałem przykładowe rewitalizacje dwóch zabytkowych obiektów. Pierwszy z nich to średniowieczne mury obronne w Trzcińsku-Zdroju, drugi obiekt to unikalna twierdza górska w Srebrnej Górze. Oba obiekty dzieli znaczna odległość geograficzna oraz fakt, iż powstały w odstępie kilku wieków. Łączy je natomiast obronny charakter obu budowli oraz determinacja lokalnych społeczności w przywracaniu dawnej świetności zniszczonym obiektom – chęć tchnięcia w stare mury nowego życia. Jednym z zadań opisanych rewitalizacji jest chęć przyciągnięcia licznych grup turystów, jak również zachęcenie okolicznych mieszkańców do poznawania tajemnic oraz historii regionów, w których mieszkają.

Oba obiekty łączy również fakt, iż do renowacji zabytkowych murów kamiennych oraz ceglanych zastosowano profesjonalne zaprawy murarskie, zaprawy do spoinowania oraz zaprawy do wzmacniania murów metodą iniekcji płynnych zaczynów. Wszystkie te zaprawy zostały wyprodukowane przez firmę quick-mix. Do ich produkcji wykorzystano historyczne receptury. Wyprodukowano je przy wykorzystaniu spoiw wapienno-trasowych powstałych na bazie trasu reńskiego Tubag.

Abstract

In the article I described exemplary revitalisations of two historical objects. The first are the medieval defensive walls in Trzcińsko-Zdrój, while the other is a unique mountains fortress in Srebrna Góra. Both objects are separated by geographical distance, as well as the fact that they were erected within the space of several centuries. However, they are linked by the defensive character of both constructions, and the determination of local communities to have the ruined objects restored to their former glory – a desire to breathe new life into the old walls. One of the aims of the revitalisations described here is the wish to attract tourists, as well as to encourage local residents to get to know mysteries and the history of the regions they inhabit.

Both objects are also linked by the fact that professional binding mortars and those for strengthening the walls by injecting cement slurry were used for renovating the historical stone and brick walls. All the mortars had been produced by the quick-mix firm according to historical recipes. They were made using lime-trass binders prepared on the basis of the Tubag Rhein trass.

Tomasz Nowak*

VIII Międzynarodowa Konferencja SAHC 2012

VIII Międzynarodowa Konferencja SAHC 2012 (Structural Analysis of Historical Constructions) odbyła się w dniach 15 – 17 października 2012 roku we Wrocławiu, w Polsce, we Wrocławskim Centrum Kongresowym przy Hali Stulecia, wpisanej na Listę Światowego Dziedzictwa UNESCO. Przewodniczącym konferencji był prof. Jerzy Jasieńko z Instytutu Budownictwa Politechniki Wrocławskiej. Główni organizatorzy konferencji to: Instytut Budownictwa Politechniki Wrocławskiej, Stowarzyszenie Konserwatorów zabytków oraz Fundacja Doliny Pałaców i Ogrodów Kotliny Jeleniogórskiej.

Konferencja SAHC odbywa się co dwa lata w różnych miejscach na świecie (poprzednio m.in. w Rzymie, Barcelonie, Guimarães, Padwie, Bath, New Delhi, Szanghaju) i jest jednym z najbardziej prestiżowych, międzynarodowych wydarzeń, które łączy inżynierów, konserwatorów, chemików, producentów materiałów, architektów, projektantów, menedżerów, naukowców i pracowników akademickich omawiających najnowsze osiągnięcia w dziedzinie teorii, analizy, ochrony oraz doktryn obejmujących konstrukcje zabytkowe. Uczestnikami Konferencji byli wybitni znawcy tematyki.

W konferencji uczestniczyło około 400 osób, w tym większość z zagranicy, m.in. z Belgii, Bośni i Hercegowiny, Brazylii, Chin, Chorwacji, Cypru, Czech, Danii, Finlandii, Francji, Grecji, Hiszpanii, Holandii, Indii, Iranu, Irlandii, Japonii, Korei Płd., Meksyku, Niemiec, Norwegii, Peru, Portugalii, Puerto Rico, Rumunii, Serbii, Słowacji, Słowenii, Stanów Zjednoczonych, Szwajcarii, Szwecji, Tajlandii, Turcji, Wielkiej Brytanii, Włoch, Zjednoczonych Emiratów Arabskich.

Ponadto uczestnikami Konferencji byli m.in.:
Bogdan Zdrojewski, Minister Kultury i Dziedzictwa Narodowego,
Rafał Dutkiewicz, Prezydent Wrocławia,
Adam Grehl, Wiceprezydent Wrocławia,
Piotr Żuchowski, Sekretarz Stanu w Ministerstwie Kultury i Dziedzictwa Narodowego, Generalny Konserwator Zabytków,
Maciej Klimczak, Podsekretarz Stanu w Kancelarii Prezydenta RP,
Tadeusz Więckowski, Rektor Politechniki Wrocławskiej,
Henryk Gulbinowicz, Kardynał Senior.

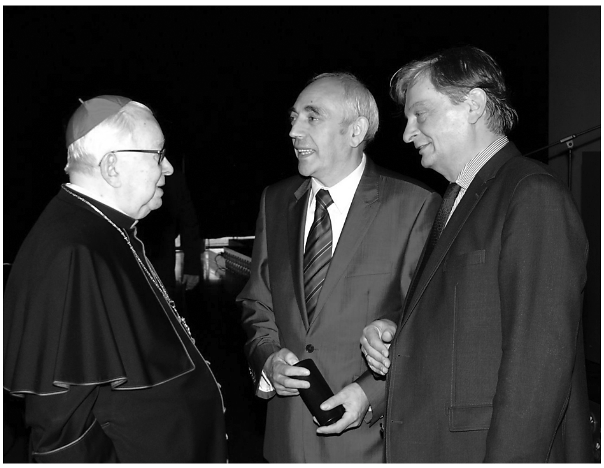
W 3 tomach materiałów konferencyjnych, wydanych w języku angielskim, opublikowano 345 referatów. Wybrane przez Komitet Naukowy Konferencji i redaktorów naczelnych czasopism referaty zostaną opublikowane w następujących czasopismach naukowych:

International Journal of Architectural Heritage,
Archives of Civil and Mechanical Engineering (ACME),
Wiadomości Konserwatorskie.

Podczas Konferencji zostały zaprezentowane również technologie związane z analizą, wzmacnianiem, konserwacją i remontami obiektów zabytkowych.

W trakcie Konferencji odbyło się posiedzenie Komitetu Naukowego International Scientific Committee for Analysis and Restoration of Structures of Architectural Heritage – International Council on Monuments and Sites (ISCARSAH – ICOMOS).

Więcej informacji na temat Konferencji dostępnych jest pod adresem: http://sahc2012.org/



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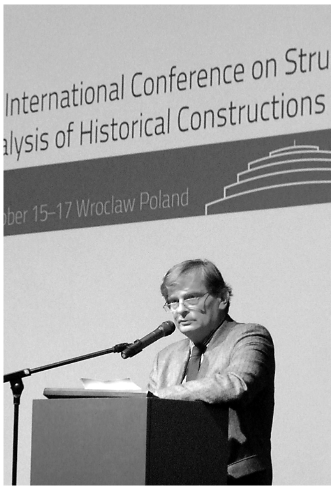
International Conference on Stru
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ber 15–17 Wroclaw Poland

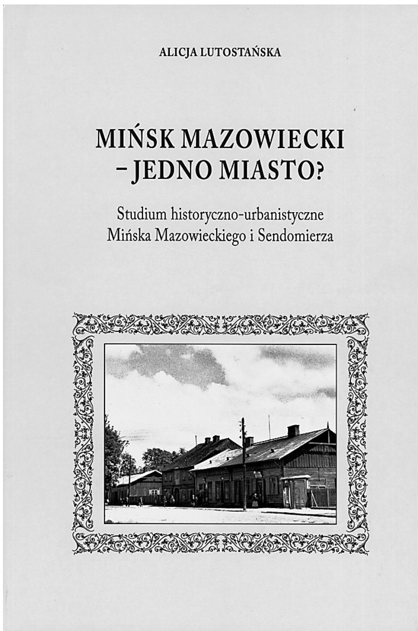
Anna Kalina-Gagnelid*

Od poszytu do albumu
From files to an album

Alicja Lutostańska, *Mińsk Mazowiecki – jedno miasto? Studium historyczno-urbanistyczne Mińska Mazowieckiego i Sendomierza.* Format 22 × 30,5 cm, ss. 103, fot. cz.-b. 53, il. barwne 3, plany 16, fragm. map 2, Towarzystwo Przyjaciół Mińska Mazowieckiego, Mińsk Mazowiecki 2012

Alicja Lutostańska, *Mińsk Mazowiecki – one town? A historical-urban study of Mińsk Mazowiecki and Sendomierz.* Format 22 × 30.5 cm, pp. 103, b-w photos 53, colour fig. 3, plans 16, map fragm. 2, Towarzystwo Przyjaciół Mińska Mazowieckiego, Mińsk Mazowiecki 2012

Studium historyczno-urbanistyczne do planu zagospodarowania przestrzennego miasta Mińska Mazowieckiego opracowane w roku 1954 przez Alicję Lutostańską zostało wydane drukiem. Autorka – dziś wybitna i zasłużona dla ochrony zabytków historyk sztuki – wówczas zaczynając swoją drogę naukową, pracowała w Pracowni Dokumentacji Naukowo-Historycznej PP PKZ. Studium to stanowiło integralną część planu stworzonego przez biuro Miasto Projekt Stolica-Wschód w Warszawie. Recenzował ją współpracujący z PP PKZ prof. Gerard Ciołek. Wydawca książki – Towarzystwo Przyjaciół Mińska Mazowieckiego – do opisywanej publikacji jako część pierwszą włączył również drugi, komplementarny tekst tej samej autorki. Jest to artykuł pt. *„Mińsk Mazowiecki – Sendomierz”*. Po raz pierwszy drukiem ukazał się on w Pracach Instytutu Urbanistyki i Architektury, Warszawa 1957, z. 2. Całości nadano tytuł *Mińsk Mazowiecki – jedno miasto? Studium historyczno-urbanistyczne Mińska Mazowieckiego i Sendomierza*.



W wyniku działań redakcyjnych maszynopis od formy poszytu przeszedł transformację, w wyniku której powstał album charakteryzujący się wysoką kulturą graficzną. Cóż sprawiło, że z tysięcy opracowań powstałych w ramach prac PP PKZ właśnie to doczekało się publikacji? W roku 2001 PP PKZ uległo komercjalizacji a jego imponujący dorobek w postaci 750 mb archiwaliów na drodze użyczenia trafił do Krajowego Ośrodka Badań i Dokumentacji Zabytków, później przekształconego w Narodowy Instytut Dziedzictwa. Do opracowania poświęconego Mińskowi dotarł jeden z członków TPMM. Nie był przygotowany na to, że otrzyma do rąk materiały będące jakby makietą do nieznanego wydawnictwa na temat historii Mińska.

Zachwycony odkryciem, fachowym ujęciem tematu, aktualnością spostrzeżeń, warstwą ilustracyjną w postaci licznych nieznanych wcześniej fotografii, postanowił wydać studium w postaci książki. Towarzystwo, mające już na swoim koncie publikacje, zaangażowało się w to przedsięwzięcie. Jego finalizacja możliwa była między innymi dzięki pomocy lokalnych sponsorów jak i przychylności dyrektor NID-u, która wydała zgodę na nieodpłatne skorzystanie z egzemplarza przechowywanego w archiwum NID. Ruszyły prace redakcji wspierane w kwestiach merytorycznych przez wybitną varsavianistkę Bożenę Wierzbicką.

W pierwotnym założeniu rolą niniejszego studium było zbadanie i opisanie historycznych warunków przestrzennych, w jakich miały następować odbudowa i dalszy rozwój miasta. Należy podkreślić, że aktualna analiza urbanistyczna Mińska wskazuje, iż sformułowane wówczas przez autorkę wnioski i zalecenia konserwatorskie do rewaloryzacji były i są nadal brane pod uwagę. W tym przypadku, kiedy to większa część historycznej tkanki podwójnego miasta nie istnieje, szczególnie ważne było zachowanie rozplanowania i założenia urbanistycznego. Wskazuje ono na średniowieczne i renesansowe korzenie. Skąd zatem ten podwójny rodowód?

Efektem wysiłków badawczych Alicji Lutostańskiej było potwierdzenie interesującej tezy o renesansowych śladach w układzie przestrzennym Mińska. Wcześniej zwrócił na nie uwagę prof. Stanisław Herbst. Co więcej, badania wykazały, że

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obecne miasto ewoluowało z dwóch odrębnych organizmów miejskich, które połączyły się dopiero w XIX wieku. Po raz pierwszy o autonomii Sendomierza wspomniał inż. Marian Benko.

Autorka na podstawie badań terenowych i archiwalnych zebrała wystarczającą ilość danych, by to potwierdzić. Dawne Mensko lokowane w średniowieczu (1421 r.), rozwijając się z pierwotnej osady targowej wchłonęło prywatne, renesansowe miasto Sendomierz. Inicjatorem jego założenia był najprawdopodobniej Mikołaj Wolski herbu Półkozic, kasztelan sandomierski. Przywilej lokacyjny przyznany został jemu wraz z synami – Stanisławem i Zygmuntem. Lokacja ta jest starsza niż powstanie Głogowa Małopolskiego (1570 r.) czy Zamościa (1580 r.). Dokonał jej w roku 1549 Zygmunt August. Rynki obu miast położone są w odległość zaledwie 200 m, na przeciwległych brzegach niewielkiej rzeki Srebrnej. Rynek sendomierski wskazuje na świadomą kompozycję planu. Natomiast rynek miński powstał na bazie owalnicy, wewnątrz której od południa wykreślony został prostokątny rynek.

Niedługo minie 60 lat od momentu powstania studium. W międzyczasie stan badań uległ zmianom, a wiele źródeł kiedyś niedostępnych ujrzało światło dzienne. Dotyczy to m.in. sendomierskiego pałacu Doria-Dernałowiczów, który nazwę zawdzięcza ostatnim właścicielom. Na przestrzeni 500 lat był on kilkukrotnie przebudowywany. Historia tych zmian nie była znana autorce. Niezależnie od tego niniejsze studium jest głosem w badaniu wkładu polskich feudałów w budowę miast renesansowych na prawobrzeżnym Mazowszu. Natomiast fakt, że analiza urbanistyczna dotyczy początku lat 50. XX wieku stanowi o jego wartości dokumentalnej. Istotną rolę grają tu wykonane w latach 1953-54 fotografie Jerzego Szandomirskiego, których 51 zamieszczono w publikacji. Ukazują one Mińsk, którego już nie ma. Ukazują miasto, w którym upłynęła młodość odchodzącego powoli pokolenia mińszczan. Stanowią one poruszające świadectwo minionej epoki i jako materiał wcześniej nieznany pozwolą dopisać nowy rozdział w procesie kształtowania i dojrzewania tożsamości lokalnej. Podpisy pod zdjęciami uwzględniają dawne i nowe nazwy ulic, co wzmacnia walor dokumentalny. Wydawca do artykułu i pierwotnego studium dołączył dodatkowe materiały w postaci historycznych map i planów (m.in. na wyklejkach) pochodzące ze zbiorów AGAD, TPMM i *Atlasu Geograficznego Ilustrowanego* J.M. Bazewicza, Warszawa 1907. Opracowano również indeks osobowo-geograficzny.

Zgodnie z zapowiedziami jest to pierwszy tom z Biblioteki Towarzystwa. Miejmy nadzieję, że idea serii będzie się rozwijać. Oby jednak w przyszłości udało się uniknąć literówek, jak np. w nazwisku autora fotografii czy takiego kadrowania zdjęć, że niewidoczne staje się to co w opisie pod nim (str. 51). W publikacji zamieszczono ponadto streszczenie w języku angielskim, listę publikacji i notatkę o autorce pióra dr Bożeny Wierzbickiej, informację nt. mecenasów kultury ziemi mińskiej, życiorys zasłużonego mińszczanina Jana Kazimierza Huberta – pierwowzoru postaci Tomasza Judyma z powieści S. Żeromskiego *Ludzie bezdomni* oraz zwięzłą historię TPMM.

Streszczenie

Studium historyczno-urbanistyczne do planu zagospodarowania przestrzennego miasta Mińska Mazowieckiego opracowane w roku 1954 przez Alicję Lutostańską zostało wydane drukiem. Autorka – dziś wybitna i zasłużona dla ochrony zabytków historyk sztuki – wówczas zaczynając swoją drogę naukową, pracowała w Pracowni Dokumentacji Naukowo Historycznej PP PKZ. Studium to stanowiło integralną część planu stworzonego przez biuro Miasto Projekt Stolica-Wschód w Warszawie. Recenzował ją współpracujący z PP PKZ prof. Gerard Ciołek. Wydawca książki – Towarzystwo Przyjaciół Mińska Mazowieckiego – do opisywanej publikacji jako część pierwszą włączył również drugi, komplementarny tekst tej samej autorki. Jest to artykuł pt. *Mińsk Mazowiecki – Sendomierz*. Całości nadano tytuł *Mińsk Mazowiecki – jedno miasto? Studium historyczno-urbanistyczne Mińska Mazowieckiego i Sendomierza*. Po raz pierwszy drukiem ukazał się on w „Pracach Instytutu Urbanistyki i Architektury", Warszawa 1957, z. 2. Całości nadano tytuł *Mińsk Mazowiecki – jedno miasto? Studium historyczno-urbanistyczne Mińska Mazowieckiego i Sendomierza*.

Abstract

A historical-urban study for the spatial development plan of the town of Mińsk Mazowiecki prepared in 1954 by Alicja Lutostańska, has finally been published. Its author – today a distinguished art historian who rendered great service to monument preservation – but then barely starting her scientific career, worked in the Department of Scientific and Historical Documentation of PP PKZ. The study constituted an integral part of the plan drawn by the office Miasto Projekt Stolica-Wschód in Warsaw. It was reviewed by professor Gerard Ciołek who cooperated with the PP PKZ. The Publisher – Towarzystwo Przyjaciół Mińska Mazowieckiego (the Society of Friends of Mińsk Mazowiecki) – included another complementary text by the same author as the first part of the above described publication. It was the article entitled: Mińsk Mazowiecki – Sendomierz. The whole was entitled *Mińsk Mazowiecki – one town? A historical-urban study of Mińsk Mazowiecki and Sendomierz*. For the first time it was published in the Works of the Institute of Urban Planning and Architecture, Warszawa 1957, vol. 2. The whole was entitled *Mińsk Mazowiecki – one town? A historical-urban study of Mińsk Mazowiecki and Sendomierz*.

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