Performance of Drilled Shaft Foundations in Limestone, Nashville, Tennessee

by Dan Brown, P.E., Ph.D.
and Robert Thompson, P.E.

Introduction

The Southeastern Chapter of ADSC has sponsored an ongoing research project to improve design methods and cost-efficiency of rock-supported drilled shafts in the southeastern U.S. In the first phase of this effort, two drilled shaft load tests have been performed at a limestone site near Nashville, Tennessee. The tests were intended to measure the performance of drilled shafts in rock that is representative of the lower bound conditions that might be expected for foundations of this type. The hope is that the test data will provide

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the basis for improvements in the design methodology used for drilled shafts in Nashville and similar major drilled shaft markets in the southeastern U.S., particularly with respect to foundations on rock.

Figure 1 Rock from Coring Tool Used to Excavate Socket for Test Shaft 1.

The testing program is also intended to serve as a test case for further research with rock-bearing drilled shafts in other types of geologic conditions.

This article summarizes the results and potential applications of the Nashville tests. The complete report of this phase of the research, "Load Testing of Drilled Shaft Foundations in Limestone, Nashville, TN" (Brown, 2009), can be downloaded by clicking on the **"Nashville Site Report" link at http://danbrownandassociates.com/research-projects.** Detailed results of the site investigation, load test data, analyses and interpretation of the load tests, and a discussion of potential improvements to design methodologies are presented in the report.

Geotechnical Conditions

After making borings to evaluate several possible test sites, a site was selected with the desired qualities, i.e., rock that was on the lower bound of typical Nashville conditions. The site was located at the equipment yard of Long Foundations* in Hermitage, Tennessee on the east side of Nashville. According to USGS geologic maps, this site is underlain by Carters Limestone of the Stones River Group, a fine-grained, yellowish-brown limestone with thin beds of bentonite clay. This formation is typical of the Central Basin limestones in the Nashville area. A photo of the rock from the coring tool used to excavate the rock socket of Test Shaft 1 is provided in Figure 1.

Soil borings were drilled in the area of the two test shaft locations as illustrated in Figure 2. The two shafts were approximately 30 feet apart, with the borings approximately 8 ft from the center of the test shafts. The data from the borings suggest that the rock for 3 diameters below the base of Test Shaft 2 (TS2) was somewhat more sound with average percent recovery

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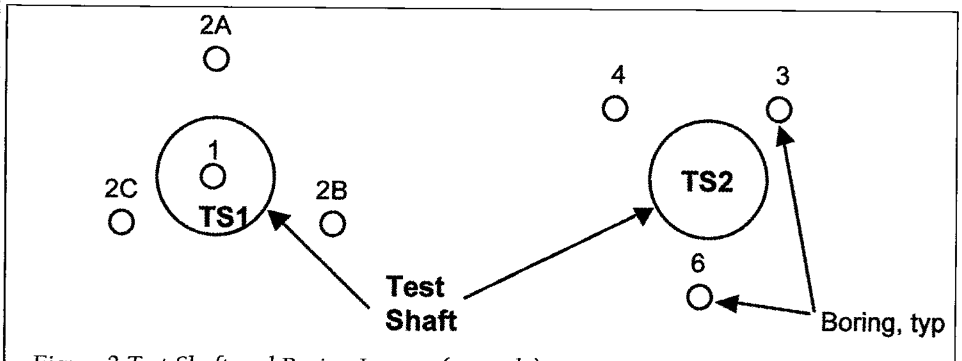

Figure 2 Test Shaft and Boring Layout (no scale).

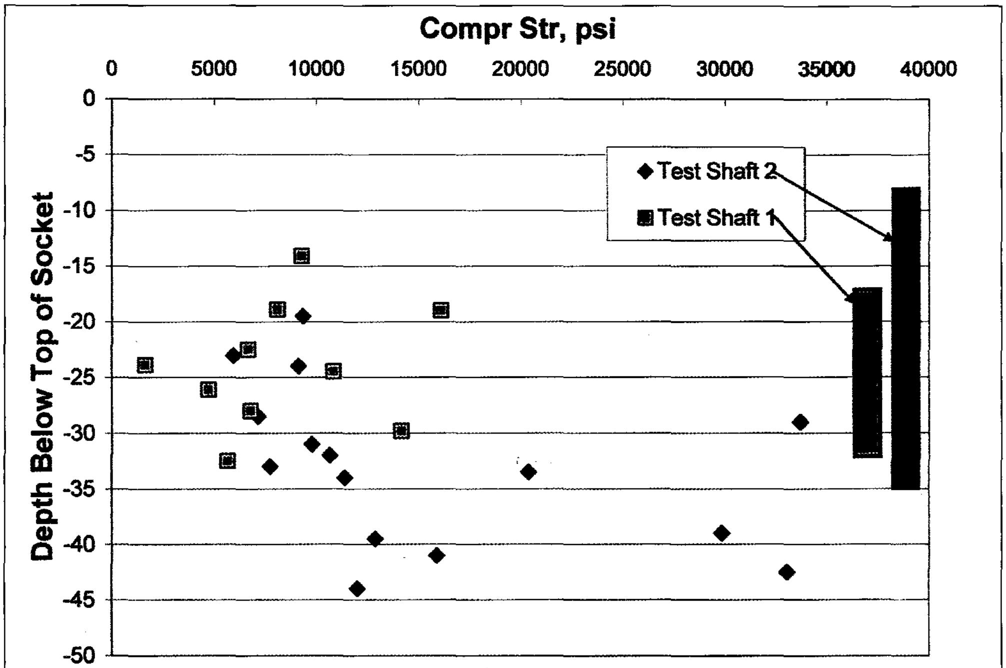

Figure 3 Compressive Strength Data from Rock Core Samples.

and RQD of 97% and 60% vs average values of 82% and 46% for Test Shaft 1 (TS1). Compressive strength data were quite variable, typically ranging from 5,000psi to over 20,000psi. A schematic diagram of the rock strength measurements from the cores are presented in Figure 3.

Construction of the Test Shafts

The two Osterberg-cell load tests at the site were designed to attempt to measure both the maximum base resistance and side resistance within the limestone. In order to accomplish both measurements, TS1 was constructed and tested first. The measured resistance from the first test was used to resize the second test (TS2) and obtain measurements that were not achieved in the first test. The configuration of the two test shafts including the location of strain gauges are illustrated below in Figure 4.

The drilling was performed by Long Foundation Company using a Watson* crawler-mounted drill rig. Both shafts were constructed in the dry by using an auger to excavate the overburden, setting a 54 inch diameter temporary casing into the top of rock, and then excavating the 48 inch diameter socket into the rock using a coring tool. The bottom cleanout of the shaft was made first using a rock auger, with final cleanout using an earth auger. No hand cleaning of the shaft base was performed; although hand cleaning is common in this area, one objective of this study was to evaluate the effect of a less stringent cleanliness requirement.

After completion of the excavation of each test shaft, an inspection probe hole was drilled in the base of each test shaft with an air-operated percussion tool for inspection of the rock below the base of the shaft. Inspections were performed by PSI, Inc.* In addition, at least six other inspectors and engineers from local practicing geotechnical firms and the Tennessee DOT (TDOT) examined the test hole in TS1.

Due to the presence of some soil seams below the tip of the shaft in TS1, the consensus of the inspectors was that under normal circumstances the contractor would have been required to extend this shaft by at least another two feet in order to penetrate below the seams. Because one objective of this study was to evaluate relatively less favorable rock conditions, the shaft was not extended. Several inspectors also noted that the cleaning of the base was not sufficient due to scattered rock debris and cuttings that had not been removed by the auger.

No large seams were observed in the probe hole in TS2 (one small seam less than ½ inch was observed), meaning the shaft excavation would have been acceptable according to lo[illegible]ice. No seepage water was not[illegible]r shaft.

Load Test Res[illegible]

The load tests w[illegible]ted by Loadtest, Inc.* Figure 5 [illegible]h of the average unit side resi[illegible] displacement for the socket of [illegible]h data plotted based on the nomi[illegible]t diameter of 48 inches and on a s[illegible]eter adjusted for over-break bas[illegible] the concrete volume. The socket [illegible]s in somewhat better rock in th[illegible] sections of the socket and the co[illegible]tion of the socket resulted in large o[illegible]ption of concrete in the upper [illegible]s of the socket due to voids in the [illegible]. In addition to the uncertainties relating to the actual dimensions of this socket, the the limit of the O-cell capacity was reached before the test fully mobilized the side resistance. The strain gauge data from TS2 suggests that the actual side resistance of the lower portion of the socket was significantly higher than for TS1.

The side resistance data suggest that the side resistance is mobilized at a relatively small displacement of around 0.2 inches or less, and that the maximum average side shear in the socket at TS1 was around 20 ksf.

In order to compare measured base resistance for similar size area, the base resistance is plotted vs displacement normalized by base diameter on Figure 6. These data are based on the projected area from the base plate plus a distribution of 2(vert) 1(horiz) downward through the few inches of concrete below the base plate.

Preliminary Implications for Design

Base Resistance

Based on the typical local practice, the rock conditions at TS1 would not have been considered acceptable and typical design would have required the shaft excavation to continue to achieve the more favorable conditions typified by TS2. It is appropriate to consider the implications for design of both conditions, hereafter referenced as follows:

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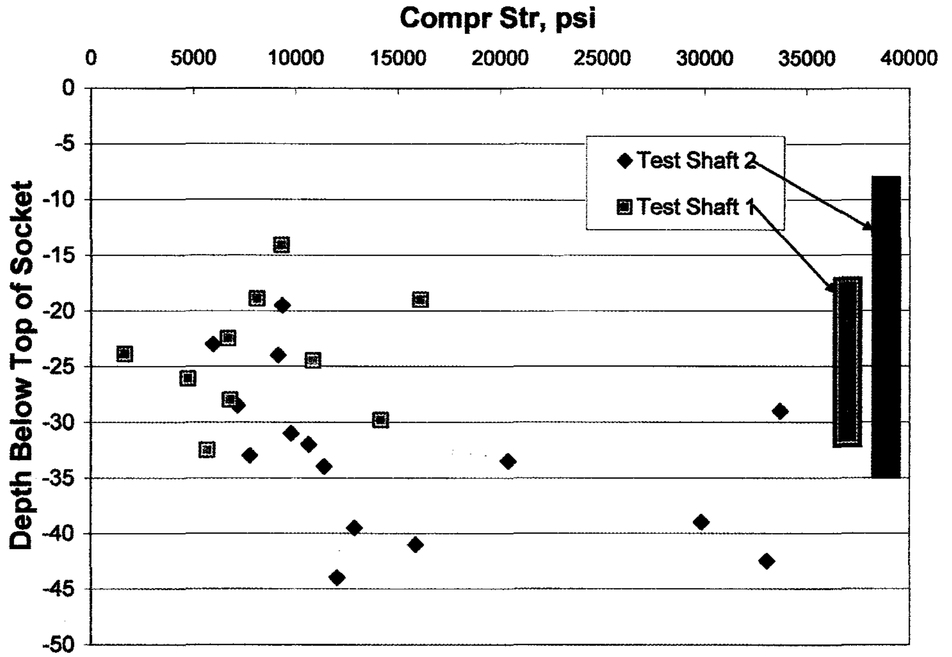

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No large seams were observed in the probe hole in TS2 (one small seam less than ½ inch was observed), meaning the shaft excavation would have been acceptable according to local practice. No seepage water was noted in either shaft.

Load Test Results

The load tests were conducted by Loadtest, Inc.* Figure 5 is a graph of the average unit side resistance vs displacement for the socket of TS1, with data plotted based on the nominal shaft diameter of 48 inches and on a shaft diameter adjusted for over-break based on the concrete volume. The socket for TS2 was in somewhat better rock in the lower sections of the socket and the construction of the socket resulted in large over consumption of concrete in the upper portions of the socket due to voids in the limestone. In addition to the uncertainties relating to the actual dimensions of this socket, the the limit of the O-cell capacity was reached before the test fully mobilized the side resistance. The strain gauge data from TS2 suggests that the actual side resistance of the lower portion of the socket was significantly higher than for TS1.

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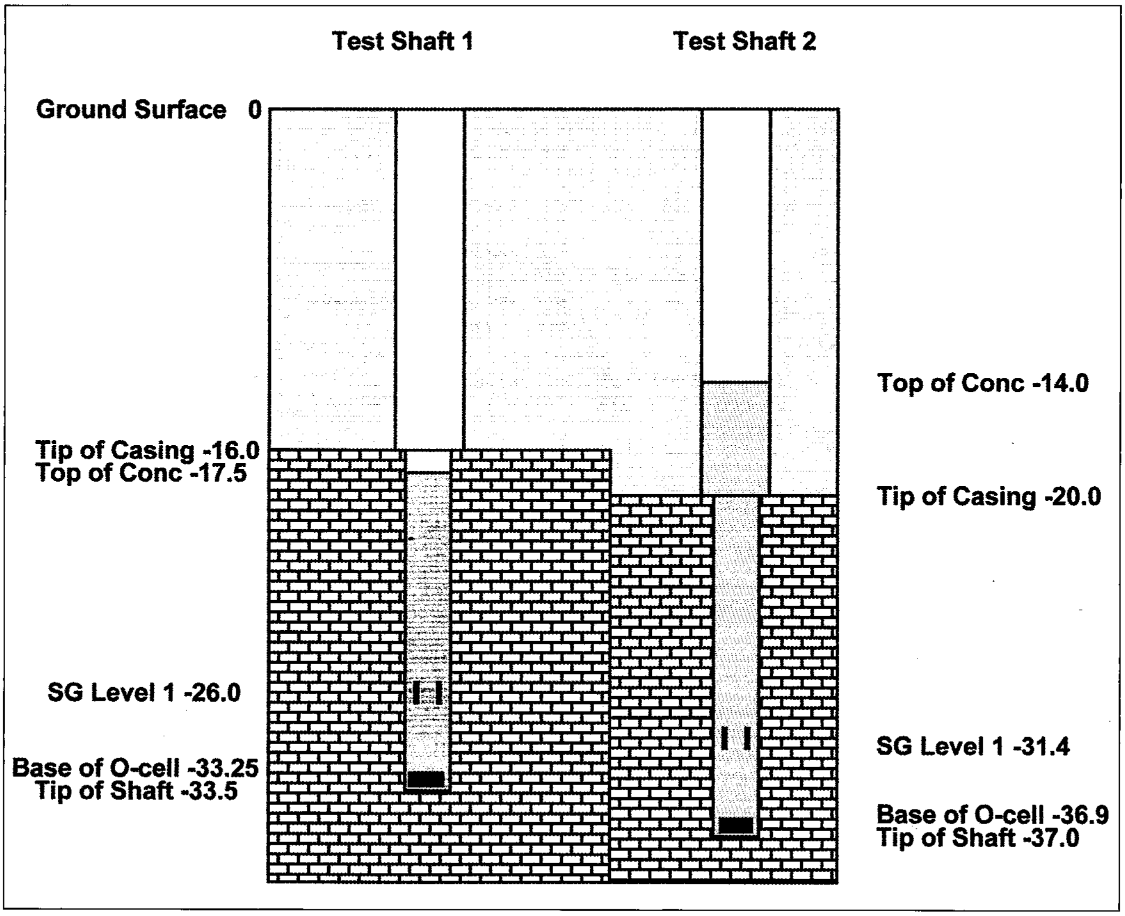

Figure 4 Schematic Diagram of Test Shafts 1 & 2.

- "Sound Rock" is typified by the conditions similar to TS2 with only one or two small seams less than ½ inch thick
- "Fair Rock" is typified by the conditions similar to TS1 with soil-filled seams up to 10% of the base diameter, B, at depths greater than ½ B.

For "Sound Rock", the test results indicate that a service load base pressure of around 500ksf (or 250 tons per square foot) could be utilized for a serviceability condition of a displacement equal to 0.5% of the shaft diameter. For this pressure, a factor of safety of 2.5 would require that the rock provide an ultimate bearing capacity of at least 1250ksf (625 tons per square foot). This bearing pressure is approximately 8600psi and exceeds the compressive stress that likely could be placed on the column from a structural strength limit standpoint. Structural strength limitations at service loads would also require that higher than normal concrete strength to fully utilize these bearing pressures. There would be little reason to include side resistance in the design of a rock-bearing shaft on "Sound Rock" since little additional resistance would likely be realized.

The report lists several geotechnical and inspection requirements that would need to be met to utilize the high bearing pressures indicated for "Sound Rock," including a thorough site investigation and inspection program, as well as target rock compressive strength ranges and maximum allowable seams within the rock. **The design of drilled shaft foundations in the Nashville area using the provided recommendations would represent a significant increase in bearing pressures over historical practice in the area. For the immediate future, it is recommended that large projects with designs based on these significantly higher values include a field load test for confirmation of design values.**

For "Fair Rock", the test results indicate that a service load base pressure of around 200ksf (or 100 tons per square foot) could be utilized for a displacement equal to 0.5% of the shaft diameter. At this value, a factor of safety of 2.5 would require that the rock provide an ultimate bearing capacity of at least 500ksf (250 tons per square foot). The geotechnical and inspection requirements listed in the report for "Fair Rock" are less stringent than for "Sound Rock," particularly in the criteria for seams. As long as solution cavities are not present, the presence of seams deeper than ½ the base diameter below the bearing elevation and with thickness of up to 10% of the base diameter is considered acceptable.

Side Resistance

It may be prudent to consider the addition of side resistance to the end bearing used in the design in "Fair Rock" conditions, particularly where 10 or more feet of socket length is required to achieve the required base resistance consistent with this condition. Note that the conditions through which the two rock sockets were constructed did not consist of rock that would qualify for the "Fair Rock" condition noted for base resistance above. The rock through which the side resistance was measured was characterized by relatively low % recovery and RQD, many seams, and highly variable compressive strengths.

For side resistance, the resistance is mobilized at small displacements and the maximum value used in design is based upon a geotechnical strength condition. For design based on the conditions measured at the Nashville site, the recommended approach is to compute the nominal (limit) side resistance using the equation:

$$f_s = C \cdot p_a \cdot \sqrt{\frac{q_u}{(p_a)}} \qquad (1)$$

where q_u is unconfined compressive

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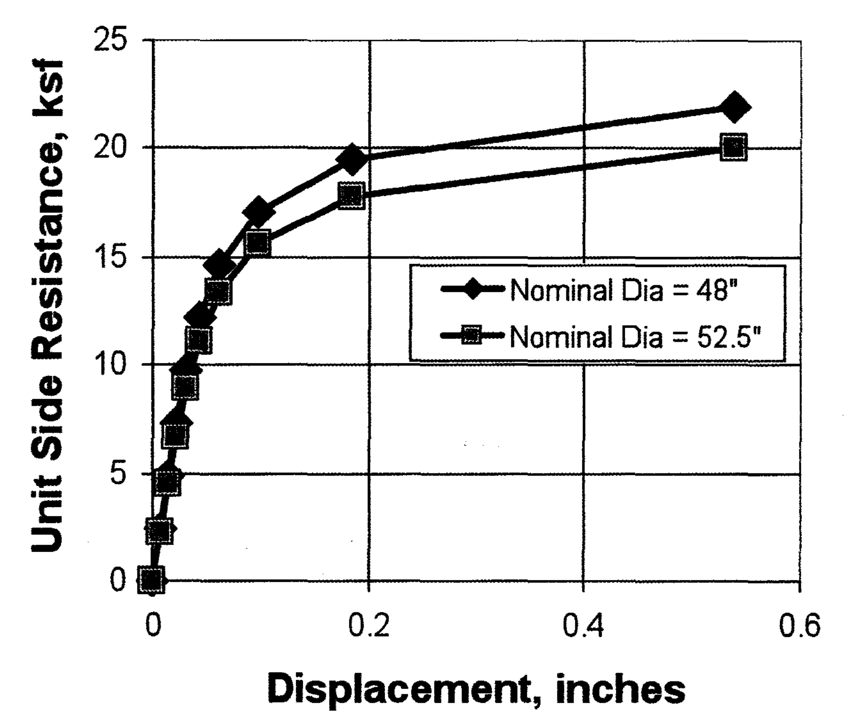

Figure 5 Average Unit Side Resistance, Test Shaft 1.

strength, pa is atmospheric pressure, and C is an empirical constant taken to be equal to 0.4, for limestone similar to the Nashville test site.

For service load conditions, the allowable side resistance may be computed using the nominal side resistance computed above divided by a factor of safety of 2.5. The allowable unit side resistance times the surface area of the rock socket may be added to the allowable base resistance to size the shaft for service loads.

It may be noted that the design based on allowable side and base resistance values will result in a greater proportion of the service load supported in side resistance because this resistance is mobilized at smaller displacements than the base. However, since the side resistance was observed to be ductile up to displacements in excess of ½ inch, the overall factor of safety is not affected by issues of strain compatibility.

Cost Implications of Improved Design in Rock

Numerous drilled shaft designs have been completed over the years using base resistance alone and an allowable (service load) design of 100ksf or less. The final report includes the consideration of several hypothetical example cases to evaluate the potential cost benefit of improved design. Unit costs believed to be typical of the Southeastern U.S. markets were used to evaluate four hypothetical projects for the cost implications of the following designs:

A. Previous practice, based on rock excavation to "Strong Rock" and end bearing alone at an allowable base resistance of 100ksf.

B. "Strong Rock" base resistance as outlined in the report.

C. "Fair Rock" base resistance as outlined in the report.

D. "Fair Rock" base resistance plus side resistance as outlined in the report.

In each of the new designs (B, C, D) a load test for confirmation of the new design values was included in the calculated costs. Design A (previous practice) does not include costs for a load test shaft.

The evaluated examples included a large structure with heavy concentrated loads, a large structure with moderate loads, and a medium structure with moderate loads. Each example compares the costs for each of the four design approaches. In all three examples, one or more of the proposed design approaches can potentially yield substantial foundation cost savings with the costs of the proposed designs being to that of the cost of previous practice, even with including the cost of a load test. The key to achieving savings is the reduced rock excavation and increased productivity associated with the shorter socket lengths resulting from increased design resistance values

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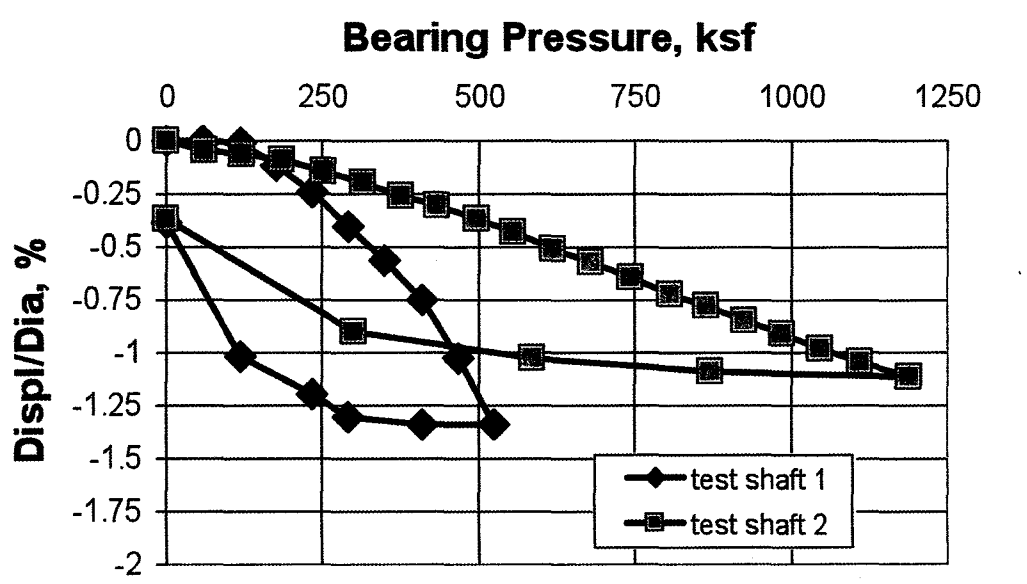

Figure 6 Base Resistance vs Displacement/Base Diameter.

and/or the less stringent acceptance criterion.

Summary and Conclusions

A program of load testing has been performed at a site near Nashville with rock that is representative of the limestone con-

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